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INLAND SEWAGE DISPOSAL, WITH SPECIAL REFERENCE TO THE EAST ORANGE, N. J., WORKS.

By CARROL PH. BASSETT, M. Am. Soc. C. E.

WITH DISCUSSION.

When the first official census of this country was taken one hundred years ago (1790) the thirteen States and unorganized territory then embraced a population of 3 929 214. Less than 5 per cent. of these people were found west of the Appalachian Mountains, the densest populations being found along the Atlantic seaboard, in Massachusetts, Rhode Island, Connecticut, and about New York and Philadelphia. The average depth of settlement at right angles to the coast was then 255 miles. The settled area comprised approximately 239 935 square miles, with an average density of 16.4 per square mile. In 1850 the total area had increased to 979 249 square miles, with a population of 23 191 876, and average density of 23.7 per mile. In 1880 the area of settlement was 1 569 570 square miles, containing a population of 50 155 783, or an average density of 32 per square mile. The center of population in 1790 was near the present site of Washington, D. C. From thence it has moved westward, about a degree to a decade, closely following the

39th parallel, until in 1850 it was a few miles south of Parkersburg, West Va., and in 1890, 8 miles south of Cincinnati.

Among the older States of the East the density of population is several times the average density throughout the country. In 1880 the average in Massachusetts was 228.6 per square mile; in Rhode Island, 211.7; in New Jersey, 135.6; in Connecticut, 131.1; in New York, 108.1; in Pennsylvania, 93.1; in Maryland, 84; and in Ohio, 80. As we examine in still greater detail the development of the country, we find in 1790 the frontier line at 255 miles from the Atlantic coast, gradually pushing inland and avoiding the arid and rugged districts. The process of natural selection makes itself felt, and the populations, having to rely almost entirely on water transportation, conform themselves to the river valleys. In our early growth, all large centers of populations were necessarily located on the larger water courses, and although the development of means for overland transportation has largely modified the conditions originally existing, experience shows that large masses of population are still most liable to appear near water-courses. A recent estimate (W. J. McGee, in the *Forum*) states that fully 25 or 30 per cent. of the population of the eastern United States is crowded upon the 14 per cent. of alluvial lowland.

If the three hundred and fifty-five cities recorded by the eleventh census in the States between New England and the one hundredth meridian are grouped according to their situation with respect to water ways, two hundred and four, with an aggregate population of 5 593 340, are found to be riparian. There are fifty-nine seaboard and lakeside cities, with an aggregate population of 6 880 043, forty-four inland cities with a population of 840 466, and twenty-eight unclassified cities aggregating 675 676 in population. Excluding the seaboard and unclassified cities, there remain two hundred and forty-eight centers, with an aggregate population of 6 033 806, of which 89 per cent. is riparian; or, including the various classes, there is a total urban population of 13 989 529, of which 40 per cent. is riparian.

Ordinarily, water for domestic and manufacturing purposes is supplied to these populations from the rivers above cities, or from a lake on the borders of which they are situated. But, from the nature of our civilization, it transpires that enormous waste products are constantly thrown off from the body corporate. Considerations of convenience have prompted the use of water-courses for the final removal of these

wastes. In the development of these tendencies we have come to see along our prominent river valleys populations pouring large quantities of filth into rivers which a few miles lower in their course furnish supplies of potable water to other peoples.

The magnitude of the rivers and the comparative sparseness of the population in past years made these conditions possible, and the evils have developed so gradually that they have escaped the attention they merit. Naturally, the rivers along the middle Atlantic seaboard furnish the best developed illustrations of pollution.

The area of watershed, and the average density of population in some river valleys in the Eastern States, is here given:

Drainage Basins.	Area in Sq. Miles.	Population in 1880.	Per Sq. Mile.
Hudson	13 248	2 280 359	172.1
Delaware	11 362	1 999 921	176.1
Susquehanna	27 655	1 715 009	62 0
Schuylkill	1 870	362 000	200.0
Passaic	949	320 762	338.0

It should be remembered that 3 miles above Philadelphia is Conshohocken, 7 miles further up the valley is Norristown, 17 miles further is Phœnixville, and 29 miles further is Pottstown; all these cities use the Schuylkill water for a public supply and also as a sewer, into which enter whatever foul liquids are voided by these cities. A few miles further up the river, Reading adds its filth, but is prevented from using the river as a water supply, owing to the presence in it of large amounts of mine water. The increase in pollution is evident from the table given on page 128.

The conditions which obtain in these instances are no exception to the general experience of the country. Chicago is wrestling with the gigantic problem. The cities along the Ohio, the Hudson and the Passaic; and in Massachusetts, along the Blackstone, Merrimac, Charles, Sudbury, Chicopee and Concord have felt the evil. Every populated river valley furnishes similar testimony. Streams acting as natural drainage channels are popularly looked upon as the outlets for crude sewage. There appears to be no adequate energy at work to change these conditions. Practically, no effort has been made in this country to

A SUMMARY OF POLLUTION OF THE RIVER SCHUYLKILL BY DOMESTIC SEWAGE, published in the Pennsylvania State Board of Health Report for 1885, from Investigation made in that year, is here inserted for illustration. (Population estimated for January 1st, 1885.)

ITEMS.	DISTRICTS.						
	FIRST.	SECOND.	THIRD.	FOURTH.	FIFTH.	SIXTH.	SEVENTH.
	Whole valley above Reading.	FROM *Upper Boundary of Reading to Mouth of Manatawny Creek.*	FROM *Above Manatawny Creek to Intake Phœnixville Water Works.*	FROM *Phœnixville Water Works to Norristown Water Works.*	FROM *Norristown Water Works to Conshohocken Water Works.*	FROM *Conshohocken Water Works to Roxboro' Pumping Station.*	FROM *Roxboro' Pumping Station to Fairmount Pumping Station.*
Drainage area	656.9 sq. m.	3980 sq. m. 1054.9 "	149.4 sq. m. 1204.3 "	517.6 sq. m. 1721.9 "	29.5 sq. m. 1851.4 "	38.5 sq. m. 1789.9 "	74.0 sq. m. 1863.9 "
Population	91 000	95 000 186 000	28 000 214 000	66 000 280 000	22 000 302 000	18 000 320 000	52 000 372 000
DOMESTIC SEWAGE—							
Daily water supply* representing domestic waste water	2 600 000 gals.	4 500 000 gals. 7 100 000 "	200 000 gals 7 300 000 "	† 500 000 gals. 8 800 000 "	1 000 000 gals. 7 800 000 "	80 000 gals. 8 880 000 "	
Population having water-closet drainage into river	5 000	12 000 17 000	750 17 750	1 100 18 850	2 800 21 650	1 100 22 750	4 150 26 900
Population having wash water drainage into river	22 000	40 000 62 000	5 000 67 000	3 000 70 000	4 000 74 500	1 500 76 000	9 000 85 000
Free ammonia	0.0035	0.0040	0.0025	0.0036	0.0010	0.0015	
Alb. ammonia	0.0140	0.0085	0.0100	0.0075	0.0140	0.0100	
Oxygen req'd	0.058	0.14	0.19	0.13	0.23	0.15	
Nitrites	....	0.00005	0.00005	0.00008	0.0005		
Nitrates	0.96	0.92	0.92	0.86	0.86	0.80	
Chlorine	0.20	0.25	0.25	0.25	0.06	0.20	
Hardness	2.5	3.5	3.70	....	....	3.0	
Total Solids	9.0	11.5	10.5	10.5	10.0	11.5	

Figures underlined indicate totals down to the lower end of the district represented by the column in which they occur.

* From public supply only.

† Perkiomen watershed above Schwenksville not included in the remainder of this column.

purify sewage where a stream of large volume offers available outlet. Present tendencies may be roughly outlined in a few words.

Many cities in populated river valleys are abandoning their river supplies, as admittedly unsafe, or are actively debating the problem of procuring water, at great expense, from distant and undeveloped highland (*e. g.*, Albany, Syracuse, Newark, Philadelphia). Inland towns, to which no neighboring stream offers possible outlet for sewage, are combining under State supervision (notably in Massachusetts), or by mutual agreement, to secure through union of interest an outlet for crude sewage into some distant water-course. It is not contended that such action is unjustifiable, or even that it is unwise under some conditions; but the practice diverts attention from different methods, in other places essential. Towns, unable to bear these expenses, are continuing on the one hand to use polluted water for potable purposes; or on the other, are storing filth in their soil and neglecting to provide sewers; because collected sewage requires some purification, which they are unwilling to attempt. Public knowledge of the true economy of stream conservation and the methods available for sewage purification, is insufficient to correct these tendencies.

With few exceptions, the laws governing stream pollution in the several States are inadequate either in their intention or methods of enforcement. A high court of New Jersey has recently endorsed the opinion that the sewage from a population of 15 000 can enter a stream having a minimum daily flow of 125 000 000 gallons, already largely polluted, and flow four miles on a level reach before entering the public water supply of 400 000 people, without demonstrated danger. The development of the germ theory of disease and the proofs on which it rests, seem likely to overturn such opinions, and it appears highly probable that demonstrated scientific knowledge will support the natural aversion of the senses, to such an assumption. Recent comparative investigations of zymotic diseases in towns using sewage-contaminated river water and those using water free from sewage inflow, furnish strong presumptive evidence that sewage-contaminated water supplies and disease are related as cause and effect. With the germ theory for a premise, no other conclusion is reasonable. It is hoped that prophylactic scientists will accumulate these evidences and press them to their legitimate conclusions.

The conclusions reached by the Water Supply Committee of the

Massachusetts State Board of Health during the past year is here relevant. "The two cities—Lowell and Lawrence—both well situated, well regulated, and comparing favorably for general healthfulness with other cities in the State, have 50 per cent. more deaths by typhoid fever, for the same population, than any other cities in the State. These are the only two cities in the State which draw their water for drinking from a river into which, within 20 miles above, sewage is publicly discharged."

The natural purifying power of streams is realized, but the safe limits of pollution are by no means understood. This limit, beyond doubt, has been reached in several streams of the country now serving as potable supplies. It is not proposed here to discuss the permissible limits of pollution in streams, or the natural processes of purification, which are actively combatting the evil. Reference is made to the following papers:*

Aside from the large number of cases where crude sewage is discharged into streams with detriment, the need of sewage purification is being felt in a growing class of inland cities isolated from any considerable bodies of water. Residence towns are likely to develop under these conditions with increasing rapidity in the future. Sewerage, if accomplished at all in these communities, must be accompanied with some purification of the sewage.

Few papers having been presented before the Society dealing with the subject in hand, the author may be pardoned for here inserting, at the expense of repeating much that is not new, some general statements regarding present practice in sewage purification, which bear indirectly on the special works here to be described at length.

Sewage purification processes, as successfully operated at present, may be grouped under the heads of—(*a*) Irrigation; (*b*) Filtration; (*c*) Chemical Precipitation, or a combination of these processes. Electrical processes are as yet barely fortified by practical experience.

(*a.*) For irrigation, the sewage is conducted in carriers or open channels and distributed, preferably intermittently, over land. Not only is

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* Charles G. Currier on "Self Purification of Flowing Water and the Influence of Polluted Water in the Causation of Disease," with its discussion, in February, 1891. Transactions Am. Soc. C. E.

Charles C. Brown on "River Pollution in the United States," before the Engineers Club of St. Louis, 1890.

Rudolph Hering, "Notes on the Pollution of Streams," read at Am. P. H. Assoc., November 10th, 1887.

purification secured, but successful utilization of the manurial components of the sewage may, in many cases, be obtained from crops raised upon the surface. When large tracts of suitable soil are available, the sale of the crops may materially reduce the cost of manipulation.

Under-drainage of the land is necessary where a compact soil is encountered. As the temperature of sewage at the outfall seldom falls below 40 degrees Fahr., little difficulty need be feared in this latitude from the freezing of the ground if it is porous and properly manipulated. The character of the sewage and the nature of the soil are important elements in determining the amount of ground necessary for irrigation. Approximately, one acre of ground for every one hundred inhabitants contributing to the sewage must be allowed, where utilization is proposed. A city of 1 000 000 inhabitants would thus require 10 000 acres. Near large centers of population such immense tracts are seldom available. At Berlin, Germany, however, which is surrounded by sandy plains, the sewage is pumped to extensive farms from 6 to 12 miles away, at Osdorf, Frederikenhof, and Falkenburg, and successfully purified. The sandy character of the ground is there well adapted to the disposal of sewage, its area is hardly limited, and the financial returns fair. Other good examples of irrigation are found; in Germany, at Dantzig and Breslau; in France, near Paris, at Gennevilliers; in England, at Croyden, Warwick, Bedford, Doncaster, Oxford, Leamington and Wrexham; in this country, at Pullman, Illinois.

Following the processes of purification in sewage subjected to land irrigation, we find the soil first acting as a mechanical filter, the coarser matters in suspension being retained on the surface. The liquid containing the dissolved matters and the smaller particles in suspension descends through the ground, and is divided into myriads of particles with an enormous aggregate surface, favorable opportunity for oxidation occurring when the soil is kept aerated. The scavenger bacteria (*bacterium termo*, most frequently), there inaugurate nitrification; oxidation or combustion of the organic impurities being coincident. Professor Warrington, in a paper read before the British Medical Association, in Montreal, in 1884, was among the first to record the result of experiments with these bacteria and note their methods of work. The function of the bacteria is to pull apart the more complex tissue of higher organisms, and resolve it into its original elements. They thrive ordinarily only to a depth of from 2 to 3 feet from the surface. It is unfortunate that

our information regarding these allies is so meagre; but careful work of investigation among able biologists in this country and in Europe, may be relied on to materially increase our acquaintance in the near future.

(*b*.) The scarcity of ground near large centers of population has suggested a modification of ordinary irrigation processes, known as "Intermittent Downward Filtration." The land to be irrigated is thoroughly under-drained at depths from 4 to 6 feet. The sewage should be applied to the land intermittently. The surface is flat or laid out on the ridge and furrow system and as even as possible, to secure uniform distribution and filtration. The sewage of about 500 people can be purified on one acre of highly porous ground properly prepared. Purification rather than utilization is the aim of this process. The process is in use in many English towns. Mr. Bailey Denton first carried the process to great perfection on a large scale at Merthyr Tydvil. In this country, the process is in operation at Lenox, Medford, South Framingham and Sherborne Reformatory in Massachusetts; Cranston, R. I.; Norristown, Pennsylvania; Morris Plains and East Orange, New Jersey.

The application of crude sewage to a small area of ground soon coats the surface with a film of putrescible matter liable to decomposition, which closes the pores of the ground. By the interception of the matters in suspension this difficulty can be avoided. For this purpose, subsidence tanks are sometimes introduced, but their effect is slow and superficial. Mechanical filters of sand, gravel, coke, charcoal, burnt earth, iron, slag, and other substances, produce results better than simple subsidence, and may be made of service for partial clarification. It is difficult to keep the media clean for efficient filtration.

Upward straining through coarse media has merits where the larger matters in suspension, only, are to be removed.

An additional modification of land irrigation has been extensively introduced in this country, from England, for the disposal of the sewage of private houses or public institutions on land immediately adjoining. The process was originally suggested by Rev. Henry Moule, in 1868; was further developed by Mr. Roger Field, in England, and was first prominently advocated in this country by Colonel G. E. Waring, Jr. Open-jointed tile are laid from 8 to 12 inches below the surface, and into these the sewage is flushed (best by an automatic syphon) and allowed to soak into the ground, oxidation occurring as in other irrigation pro-

cesses. The process is in use at many large public institutions, residences, hotels, and is advocated for the disposal of small volumes of sewage, where cesspool, storage, or direct stream pollution is otherwise liable to occur. Excellent illustrations of this system may be seen at Morris Plains and Lawrenceville, New Jersey, and Bryn Mawr, Pennsylvania.

(*c.*) It remains to consider processes of chemical precipitation. Many of the faults and objections raised against this process are as weak as were the extravagant advantages claimed for it a few years ago in England. The author's belief is that in some form of chemical precipitation lies the great hope for sewage purification, in compactly populated districts. The precipitation of matters from solutions has long been recognized as a possibility by chemists, and profitably applied in manufactures. An infinite variety of chemical precipitants for sewage treatment has been suggested, but the essential details of all are identical.

The selected chemicals are thoroughly mixed with the sewage, after which the liquid is brought to practical rest in large tanks, where precipitation occurs. The process is first mechanical, the heavier matters settling by their own gravity to the bottom of the tanks; but a second action should be secured by the chemicals, viz., the creation of a coagulating film which entangles and carries down the finer particles in suspension and removes some of the matters in solution. The impurities deposited in the tanks, together with the water retained by them, form a dark, viscous mass, containing from 90 to 95 per cent. of water, technically called "sludge." The greatest difficulty experienced in chemical precipitation is in disposing of this sludge. Absorbents are sometimes added to render it portable. The method of manipulation by filter-presses has given probably the best results, and will be described more fully herein.

Cheapness and availability of the chemicals used in precipitation are matters of prime practical importance; to these considerations is largely due the general use of lime, salts of iron, and alumina. Blood, charcoal, clay, phosphates, manganates, and chlorine are among the other chemicals that have been used. It has been found possible in England to secure an effluent from sewage purification works, where a chemical process only is used, which conforms to the standard of purity erected by the River Pollution Commissioners.

A process known in England as the "International," and approved

by Professor Frankland, uses for a precipitant a patented compound called Ferozone (above 3 grains per gallon), and the effluent filtered through "polarite" is clear and odorless; the sludge is compact, about 40 per cent. of the bulk of the precipitate being from lime. Recent experiments at Salford, England, with this process, showed results about as good as those from an electrical test, and better than those from the albumino-ferric process.

The electrical process with a 50-volt, 50-ampère current, consuming iron at the rate of .46 pounds per 1 000 gallons, produces an excellent effluent and compact sludge. For full description of the electrolysis of sewage, see *Engineering News*, October 26th, 1889, an abstract of an article read before the British Association by William Webster.

Chemical precipitation processes are in operation at Frankfort, Germany; Coventry, Leeds, Hertford, Aylesbury, Tottenham, Layton, Burnley, Bradford, Leicester, Southampton, and other places in England. In this country the first sewage purification works involving a systematic chemical treatment on a large scale, were erected at East Orange, N. J. Worcester, Mass., has since placed in operation, works purifying part of the city's sewage. At Round Lake and Coney Island, N. Y., and at Long Branch, N. J., works for partial precipitation of sewage have also been erected.

It is now proposed to describe the East Orange Works and the steps which led up to their construction. As early as 1881, the citizens of East Orange, who are largely active, intelligent business men of New York City and Newark, began to look about for some method for removing domestic wastes. The town is situated along Newark's western boundary and is surrounded by South Orange, Orange City, and Bloomfield on the south, west and north, respectively. The small streams near the town furnish no available outlet for crude sewage, and land in the midst of an exceptionally well-to-do community was difficult to procure even at high figures, particularly for a matter so æsthetically unpleasant as sewage purification. Outlet through the sewers of Newark on the east was suggested, but without approval by that city, and a pumping-main over the ridge to the south, requiring the elevation of the entire sewage, to extend eight miles across adjoining townships to Newark Bay, was designed. Reference to the accompanying plan (Plate XVII) of the thickly-settled parts of Essex County, N. J., will show the relation of

PLATE XVII
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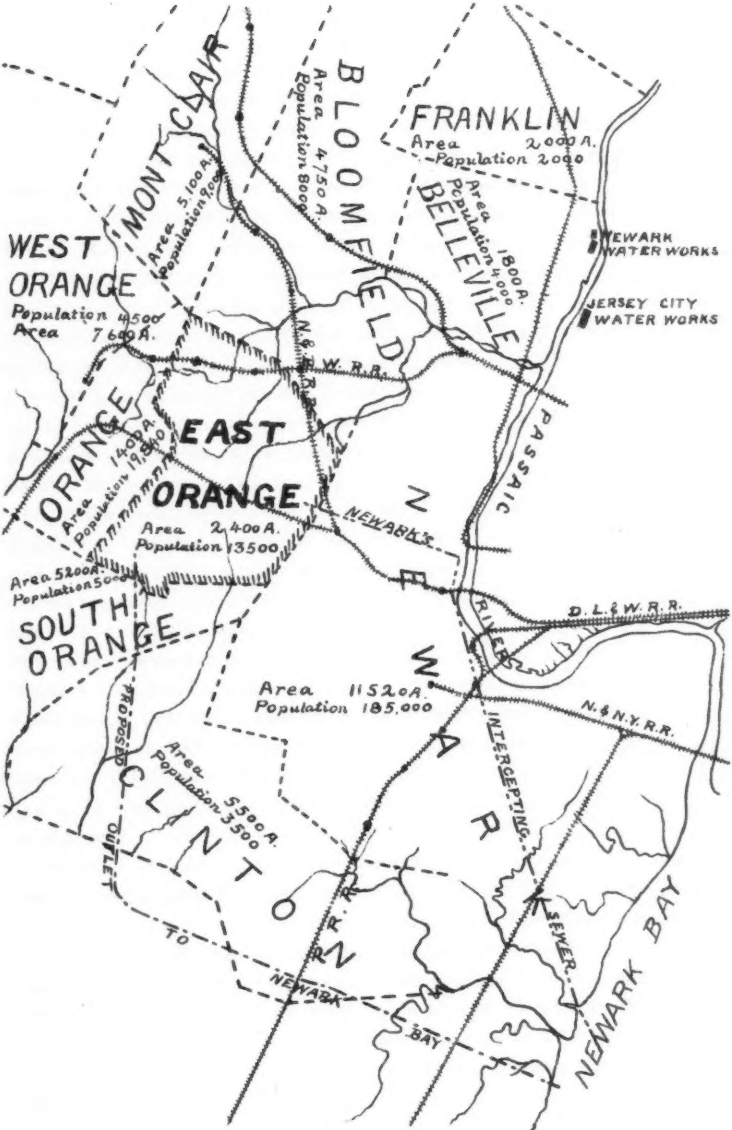

East Orange to its neighbors and the elements of difficulty involved in the problem.

After five years of earnest discussion of the various phases of the question, the author was engaged in the spring of 1886 to design the details of a plan previously submitted in outline, providing for local purification of the sewage; and works in conjunction with a separate sewerage system, embracing 26 miles of street mains, were constructed under his direction and placed in operation in June, 1888. Mr. Rudolph Hering, M. Am. Soc. C. E., reviewed the plans as consulting engineer.

The agreement to prepare detail surveys and plans was signed in the latter part of May, 1886, and so great was the pressure of the committee to begin work, that a contract for the construction of the sewerage system was made in the latter part of August following, with Mr. B. J. Coyle of Washington, D. C. The disposal works were to be constructed by day-labor under my direction. Mr. Coyle, after completing about five miles of the easier work, abandoned his contract during the winter of 1886–87, and the committee completed the construction of the balance of the system by day-labor, under the author's direction. No borings had been made to reveal the difficult character of the construction, and the unexpected constantly occurred. Heavy cuts, in some cases over 30 feet, mainly in water-bearing rock, were encountered by the collecting sewers in crossing the ridges, while in the valleys quicksand was everywhere encountered near the surface. All sewers over 8 inches in diameter (and about 4 miles of such) were constructed under one of these conditions. The utmost care was taken to secure good work; artificial foundations of timber or concrete, or both, were largely used; but no special separate underdrainage was secured or attempted. In the spring of 1888 the entire system was completed and opened for house-connections.

Mention has been made of the unusually large percentage of the sewers throughout the township which were constructed in water. In all the deep cuttings, the water level is now far above the sewers, a head-pressure of over 20 feet occurring in several places. In addition, all the mains located in the valley lines were constructed, as has been noted, in quicksand or running-sand formation. Under these circumstances, despite the greatest care and much expense, a considerable volume of ground-water finds its way into the sewer-pipes. When it is remembered that there are over 2 600 joints per mile, some of them over 6 feet

in circumference, the practical impossibility of making actually impervious sewers with vitrified pipe and cement under the conditions named, becomes apparent. But this flow from the 25 miles of pipe-sewers was limited to a small volume, probably about 2.5 gallons per second. It was necessary, however, to build the outfall sewer with a size beyond the maximum vitrified pipe, and a brick sewer was therefore constructed for 2 000 feet through a difficult formation, a timber cradle being used under the sewer. In another section of the town a tunnel was driven for about 2 000 feet at a depth varying from 25 to 35 feet, to avoid the interference with surface travel incident to so tedious a work in open cut. The great difficulty experienced in controlling the large volume of water encountered at this depth, practically prevented the construction of an impervious sewer in this place, where the tunnel was lined with brick. From these two pieces of brick sewer, less than a mile in length, about 5 gallons per second enters the sewers—twice the quantity, it will be noted, entering the remaining 25 miles of pipe-sewers. This aggregate flow of 7.5 gallons per second (650 000 gallons per day) mingles with the house sewage, becomes sewage and must undergo the purification processes. Such a flow is a material detriment to the operation of the purification works. Had information concerning the subsoil water of the township been secured—which the limited time available for the preparation of the plans prevented—underdrains to remove independently at least a large part of this flow would probably have been provided.

Plate XVIII is the general map of the township showing the sewer system as constructed.

On Plate XIX is shown the sizes and lengths of the sewers in the system.

The natural trend of the drainage of the township is to the north through four nearly parallel valleys, which break from the plateau forming the southern half of the township and dip to the north. In the northeastern district are located the water works supplying East Orange and Bloomfield. The water is here secured from shallow wells in the new red sandstone. It was expedient to keep collected sewage out of this district. It was determined to collect the sewage from approximately 2 000 acres, or five-sixths of the township area, by an intercepting sewer crossing the northern district from east to west (see Plate VII) into the Second River valley. The arguments in favor of this point of col-

MAP
EAST ORA
SHOWIN
SEWERAGE
SCAL
LEGEND
Sewers
Manholes
Flush Tanks
SOUTH ORANGE
HIGHLAND AVE.
HARRISON ST.
CLAY
ELIOT PL.
TREMONT AVE.
SANFORD ST.
BEDFORD ST.
CRAWFORD ST.
TELFORD ST.
HALSTEAD ST.
NORWOOD ST.
CLINTON
CAMBRIDGE ST.
PRINCETON ST.
EVERGREEN PLACE
AMHERST ST.
BURNETT
CENTRAL
BEECH ST.
CHESTNUT ST.
ORANGE ST.
WINANS
MAPLE
ELMWOOD
WEST ST.
OAK ST.
AVE.
LINDEN AVE.
PARK ST.
ARLINGTON
MUNN
JONES ST.
OLIVER ST.
HAWTHORN AVE.
MAPLE
MAIN ST.
MORRIS &
ESSEX R.R.
Limit of Area
GROVE ST.
PULASKI
STEUBEN
STIRLING
HEDDEN PL.


MAP OF
ORANGE N. J.
HOWING THE
RAGE SYSTEM
SCALE.
TRANS
INLAN
ORANGE
NEWARK
BLOO
NORTH
PARK ST.
E. PARK ST.
STATE ST.
HILL
WASHINGTON
BRIGHTON AVE.
NEW YORK &
MIDLAND
GREENW
GLENWOOD
AVE.
PARK
SPRINGDALE
FRANKLIN ST.
WEBSTER PL.
BALDWIN
ESSEX
PLACE
PROSPECT
MAIN
CARLETON ST.
PROSPECT
LLOYD AVE.
NORTH
CLINTON
SUMMIT
MULFORD
WALNUT
HAMILTON
WINANS
ST.
WILLIAM
AVE.
JONES ST.
NEW
THOMPSON ST.
MORRIS &
MAIN
ST.
GROVE PL.
ESSEX R.
ENWOOD
Requiring Pumping
Area
NINETEENTH ST.
EIGHTEENTH ST.
SEVENTEENTH ST.
SIXTEENTH ST.
FIFTEENTH ST.
FOURTEENTH ST.
Pumping Station
New Pumping Station
HEDDEN PL.
NEWARK & BLOOMFIELD BRANCH OF THE D. L. & W.

PLATE XVIII.
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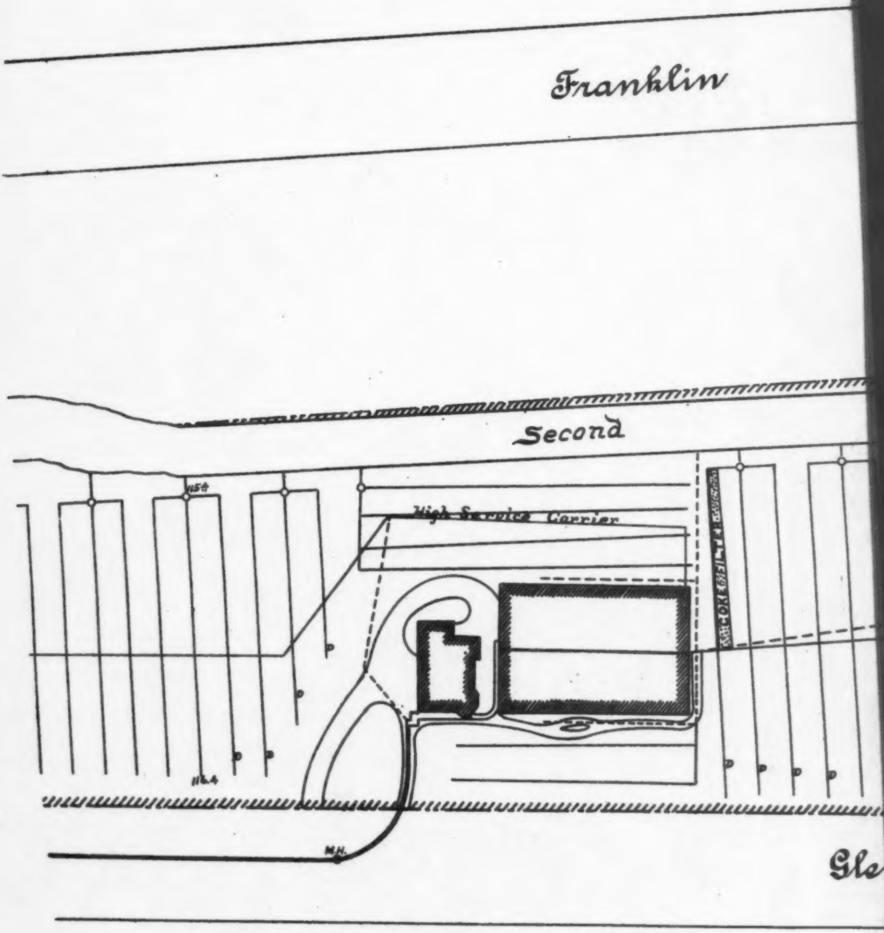
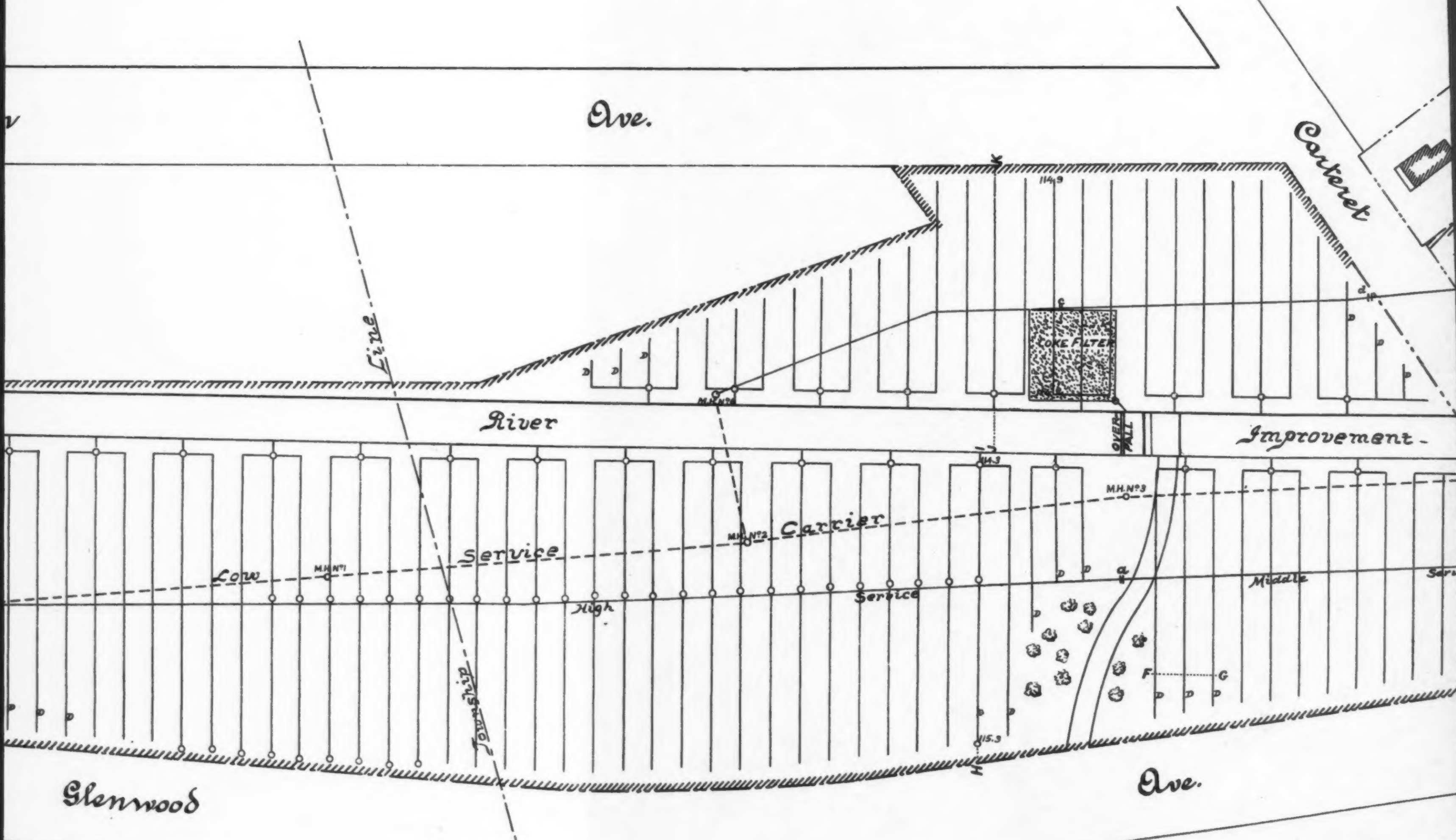

Franklin
Second
High Service Carrier
M.H.
Gle



LEGEND.

D, D.—Tile drains.

A. B. C. D.—Tumbling Steps.

Area available for filtration enclosed by hatched lines =

In East Orange, 3.5 acres.

In Bloomfield, 8.5 "

PLATE XIX
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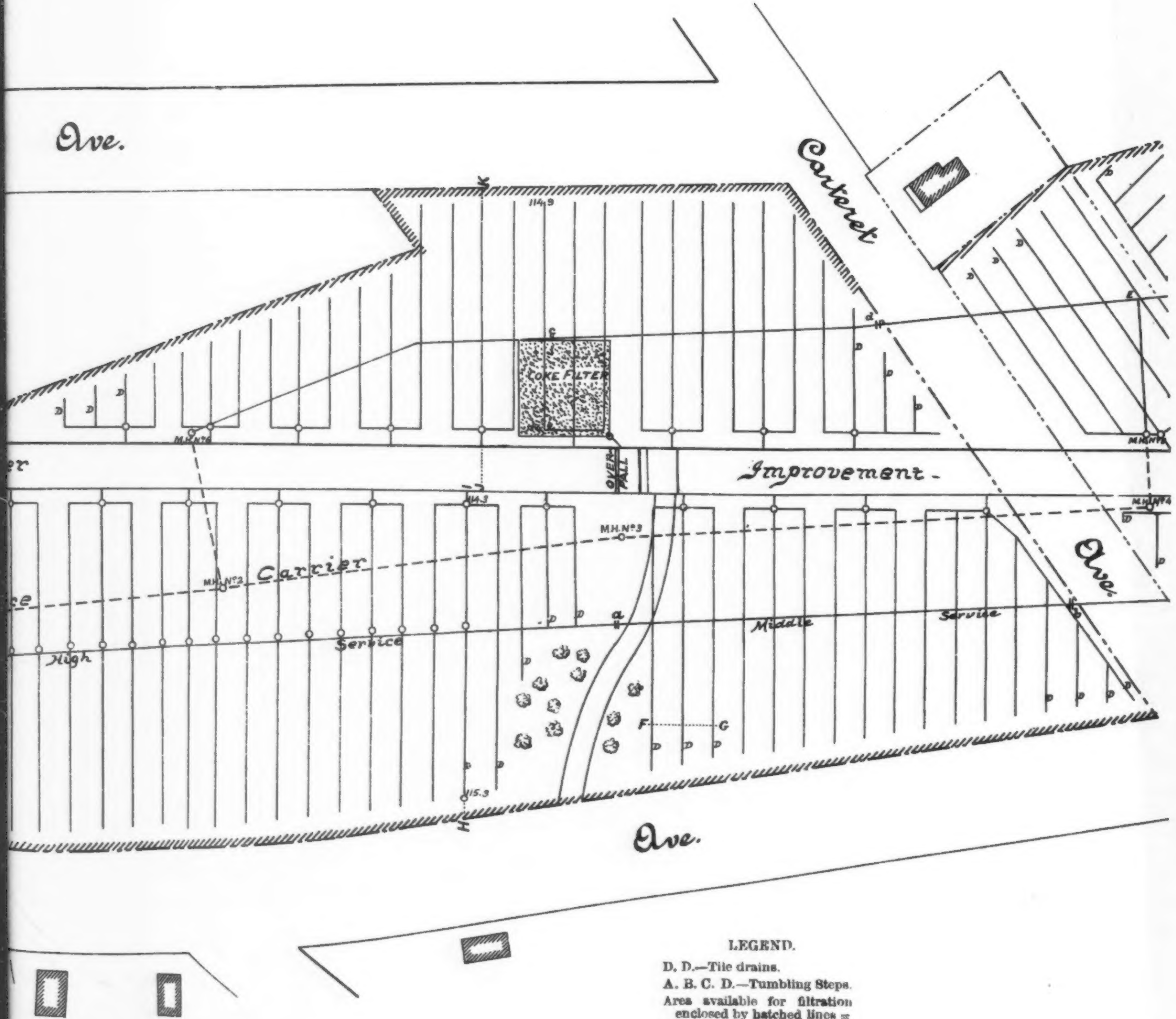
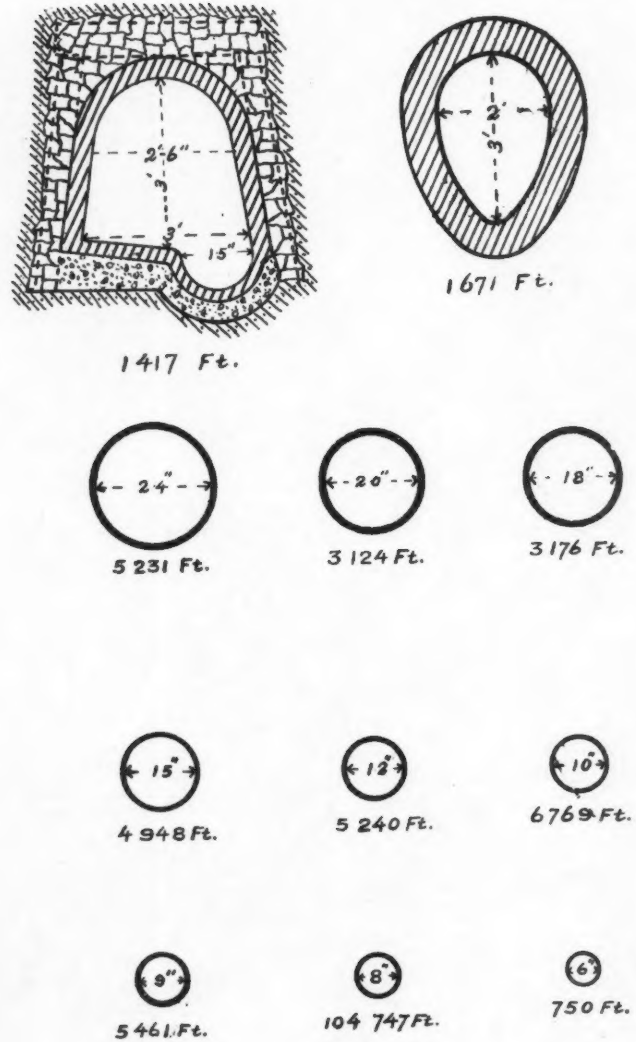
INLAND SEWAGE DISPOSAL.





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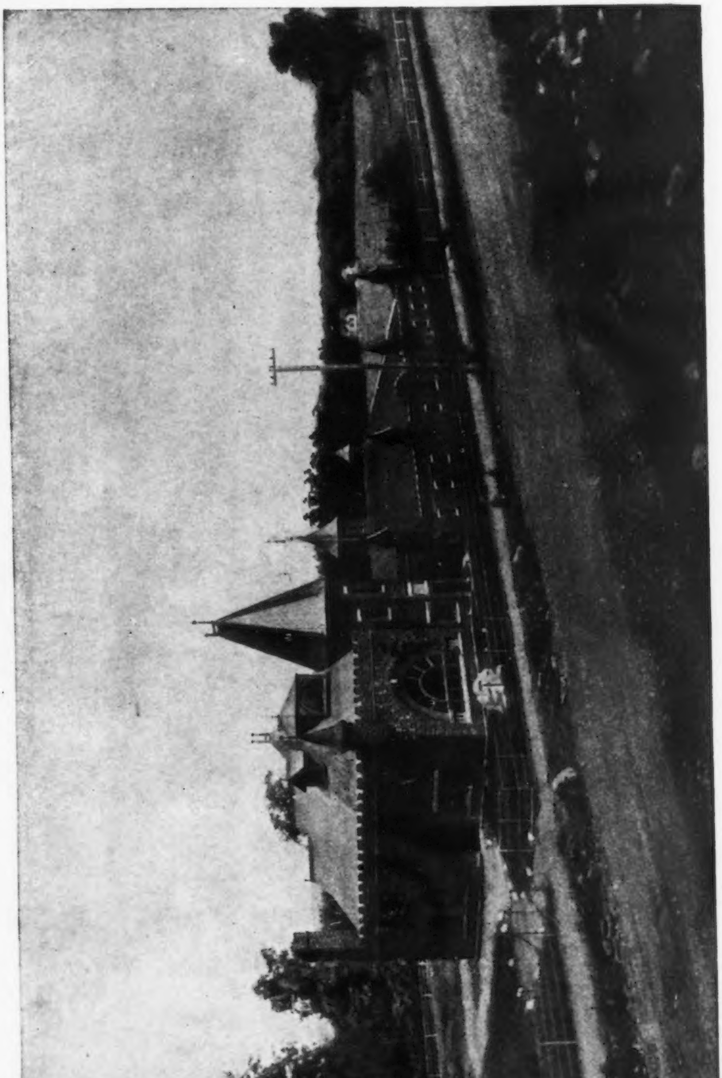
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PLATE XXI.
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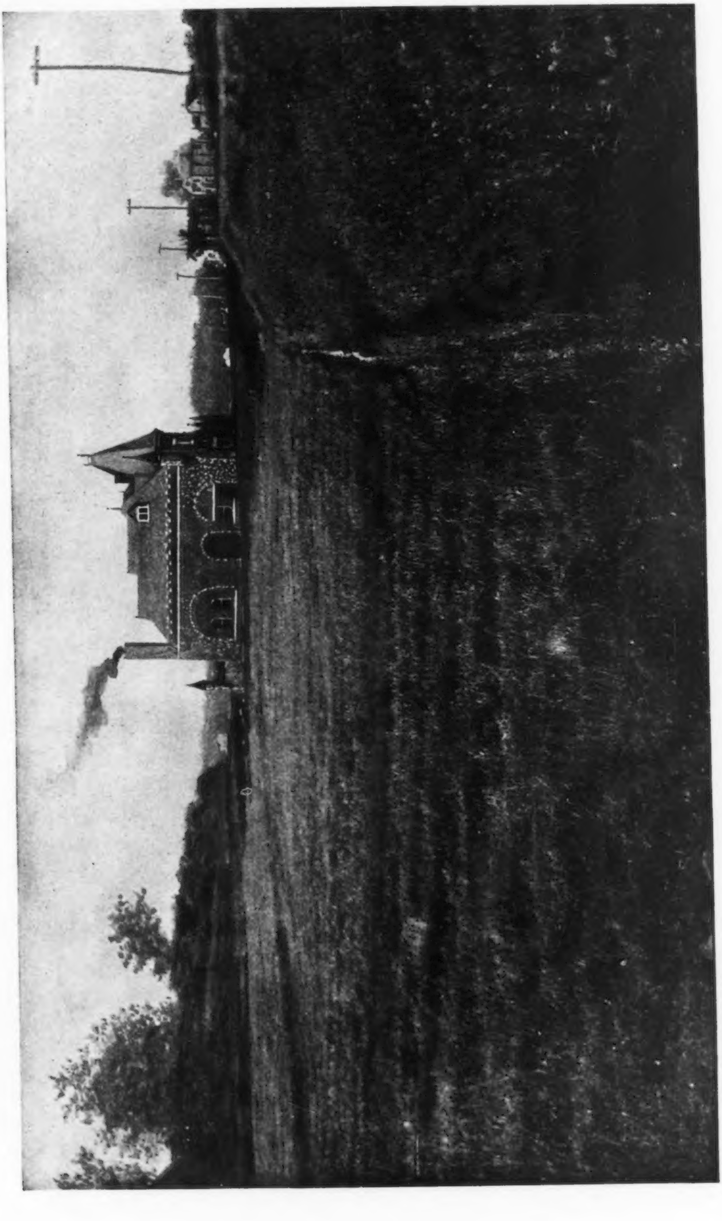
BASSETT ON INLAND SEWAGE DISPOSAL.



lection were: (*a.*) A larger percentage of the area of the township could be collected to this point by gravity, than to any other. (*b.*) The sewage would be united at the best point for ultimate gravity extension to tide-water, or combination with other towns if it were desired. (*c.*) About the only land in the township available for sewage treatment was here reached. (*d.*) The stream, offering outlet for the effluent, was larger than any other in the district.

The land secured for the works was singularly unfavorable for sewage purification. The total area available was about 15 acres; of this 5 acres were covered by Dodd's mill-pond, and the character of its bottom may be understood, when it is remembered that repeated complaint of its deposits had been made by residents to the Health authorities. The drainage and transformation of the pond was held out to hostile residents, as consolation for the location of sewage purification works in their midst. Reference is made to the general plan of the works, Plate XIX. This, together with the views shown in Plates XX and XXI, will show the residences immediately adjoining.

The stream indicated on the plan, originally fed the pond, but its channel has been deepened and straightened—a rather expensive piece of work, some of the excavation being made in quicksand. It is a tributary of the Passaic, called Second River. Its volume varies from 12 cubic feet per second, in dry weather, to 775 cubic feet per second flood volume. After a flow of about 4 miles it enters the Passaic, near the intakes for the water supply of Newark and Jersey City. Under these conditions it was necessary to secure a very high purity in any sewage effluent which was to be discharged into the stream, and the works must be operated without local nuisance. No reasonable expense was spared to make the works efficient and attractive; the buildings constituting the works are shown in the photographs Plates XX and XXI. A pleasing architectural effect is secured. The masonry is of high class; deep blue trap-rock, with rock face and worm joints, pointed with red mortar, relieved by red brick trimmings and cut stone capitals at the front about the doors and windows, secure a permanent and attractive appearance to the works.

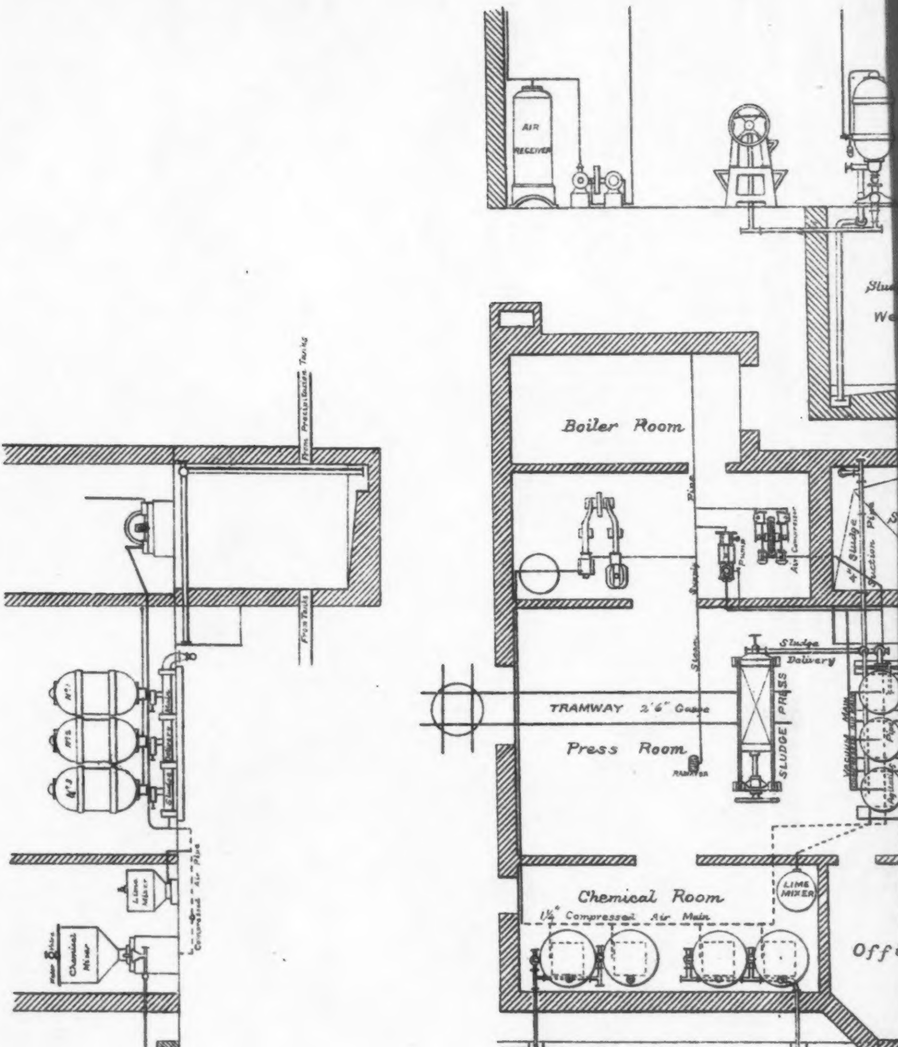
The sewage enters the works in a 2-feet by 3-feet new form, egg-shaped, brick sewer; discharges into a conduit of rectangular section, having lateral projections extending nearly to its center on alternate sides at intervals of three feet along the axis. In this conduit, chemicals

from the building join the sewage; the lateral projections of the carrier give a whirling motion to the sewage, which causes a complete mixture of chemicals with it. The carrier leads the sewage to the precipitation tanks.

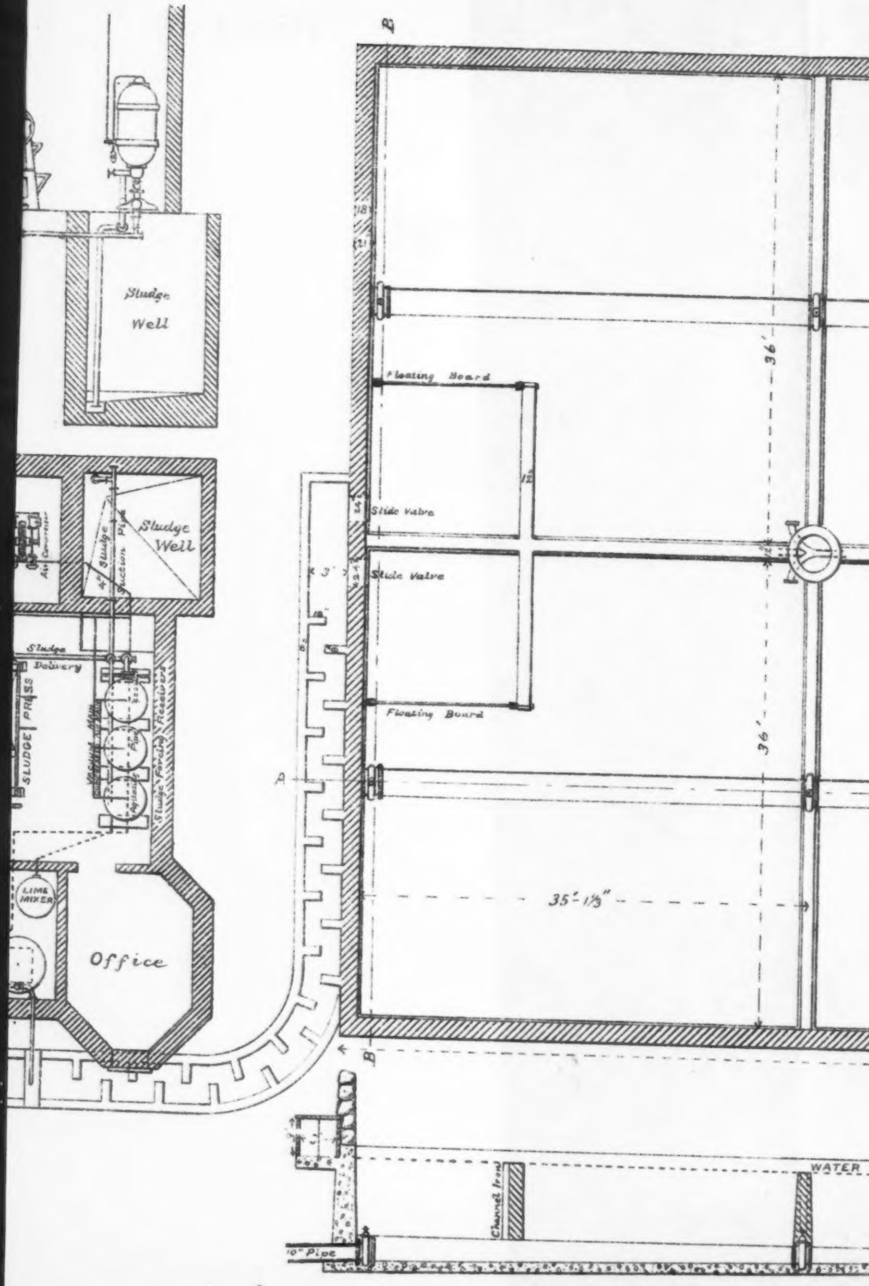
The tanks are constructed in duplicate, one set being cleaned or lying idle while the other is in use; Plate XXII gives a general plan of the building and tanks with longitudinal and cross sections. A brick wall located 10 feet in front of the inlet to the tanks, checks the velocity of entrance flow. A board floating on edge in vertical guides, intercepts the lighter floating matters, and insures their saturation before passing it. The cross-walls in each tank divide it into three compartments, and the flow passes over these with a depth of about 2 inches, the heavy matters being intercepted and settling. With a continuous flow of low velocity in the tanks, the surface water is being constantly skimmed off into the carrier. Drums float a swivel arm in each compartment, which connects with a low service pipe in the bottom of the tanks that discharges on the surface of the ground at a lower level. These arms draw water only from the surface, but the drums falling with the water enable any arm to empty the compartment in which it is located into the low-service carrier, leading to the surface of the grounds. The effluent from the precipation tanks, after entering the carriers (Plates XIX and XXIII), is distributed over the surface of the filtration grounds and descends to the under drains, which are from 3 to 5 feet deep and 20 feet apart over the entire 14.7 acres in the works.

The sewage effluent is applied to the land on the principle of intermittent downward filtration, the flow being applied successively to different areas. Part of the land is laid off in beds, 4-feet wide, separated by shallow furrows in which the water flows, and soaks laterally into the beds. The remainder of the land is divided into flat beds, 100 feet long by 50 to 100 in width, over the whole of which water flows. This latter method is preferable, as more water is disposed of, and in winter, frost is more easily kept out of the ground. Italian rye-grass has given the best results on the land, and is now grown almost exclusively. Farmers from the neighborhood cut the grass and remove it as is necessary, but up to the present time the town authorities have not been able to secure a satisfactory return on its sale.

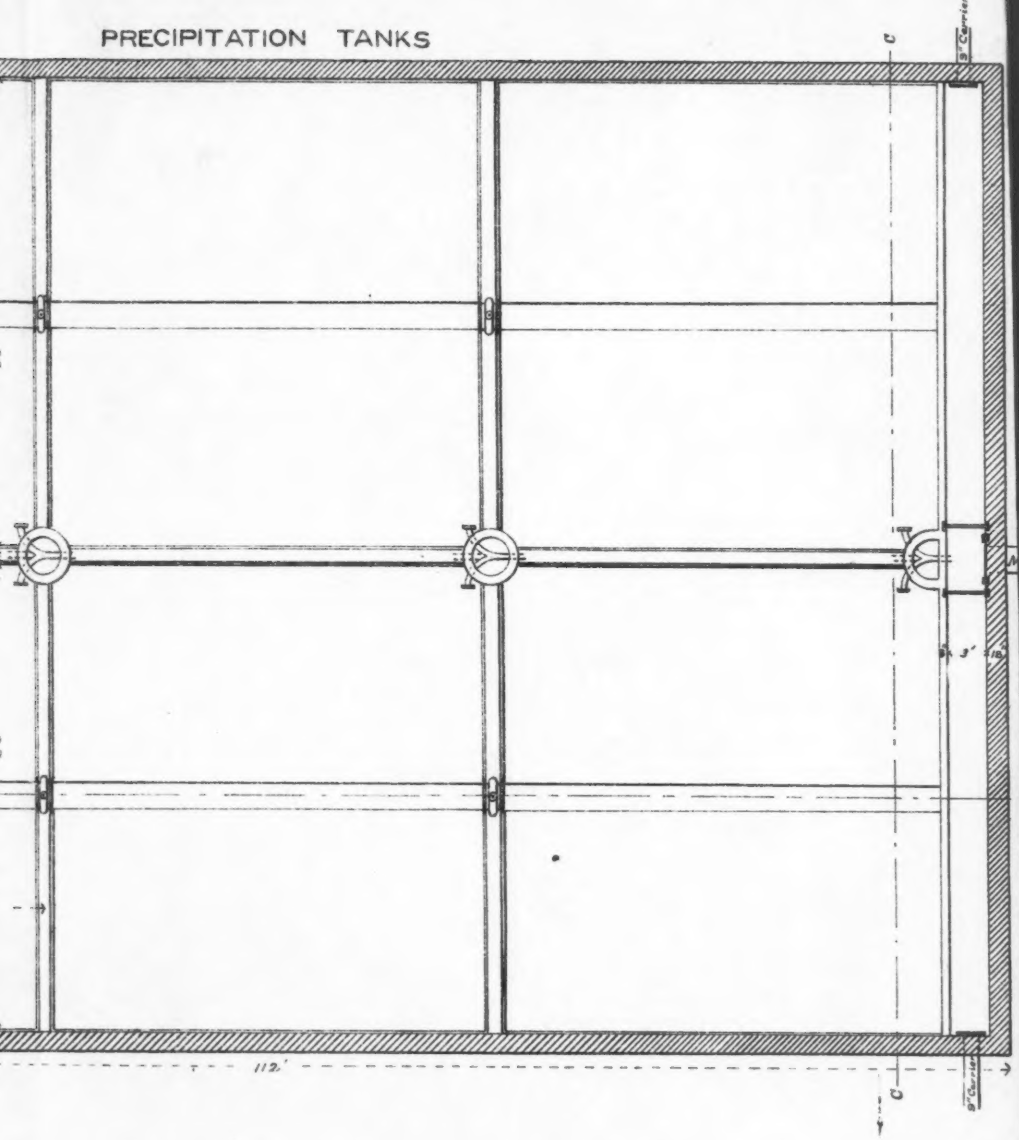
Within the main building on the first floor are chemical mixing vats, filter presses, sludge-pressing machinery (receivers, air compressor and

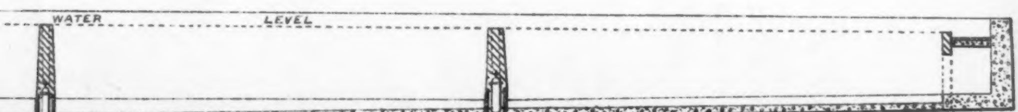


AIR RECEIVER
Boiler Room
Pump
Air Compressor
Steam Pipe
4" Sludge Suction Pipe
Sludge Delivery
TRAMWAY 2'6" Gauge
Press Room
SLUDGE PRESS
Chemical Room
1¼" Compressed Air Main
LIME MIXER
From Precipitation Tanks
From Tanks
Compressed Air Pipe
Lime Mixer
Chemical Mixer


B
Sludge Well
Floating Board
Slide Valve
Slide Valve
Floating Board
36'
36'
Sludge Well
Sludge Delivery
SLUDGE PRESS
LIME MIXER
Office
35'-1⅓"
A
B
WATER
Channel Iron
10" Pipe

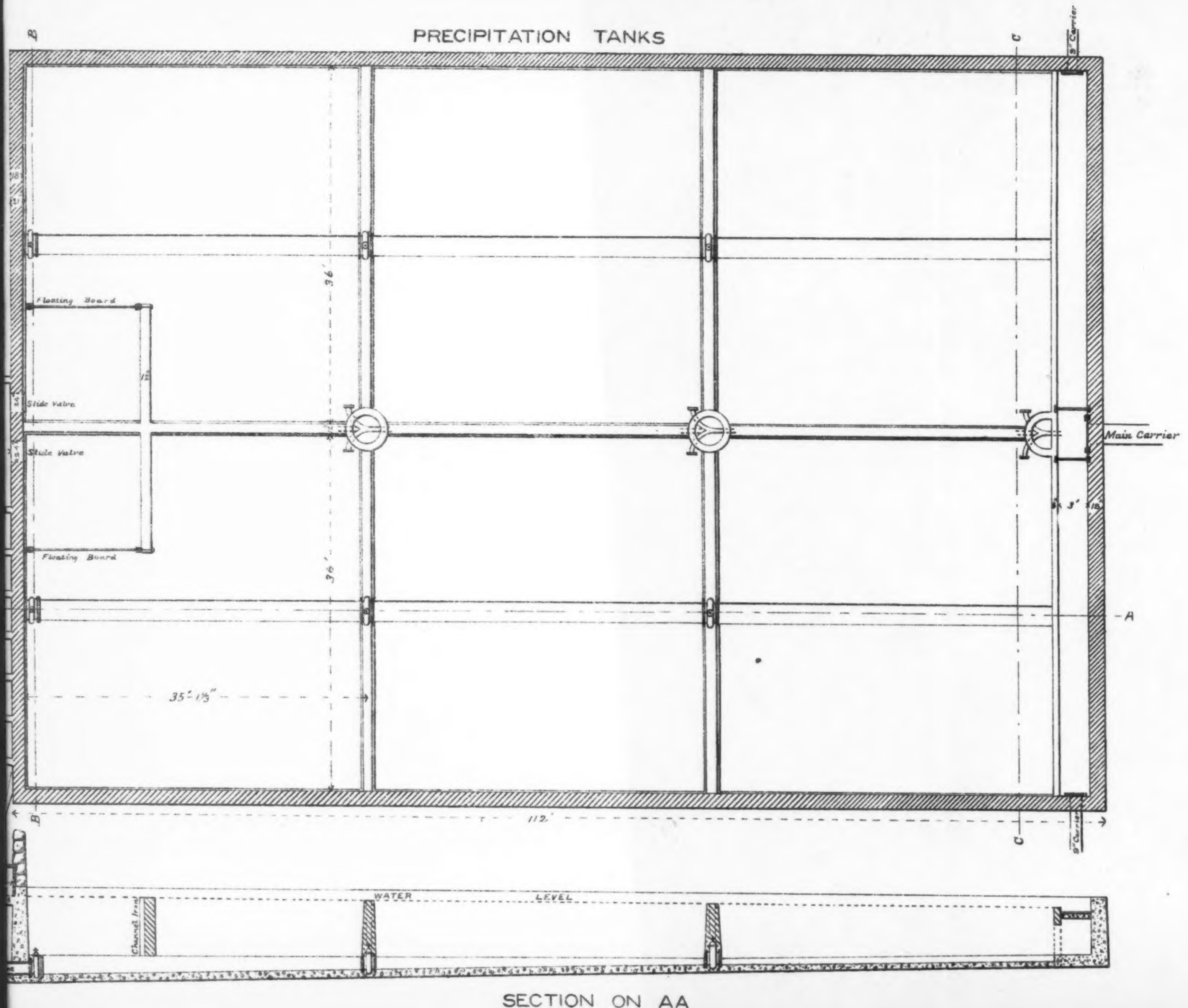
PRECIPITATION TANKS

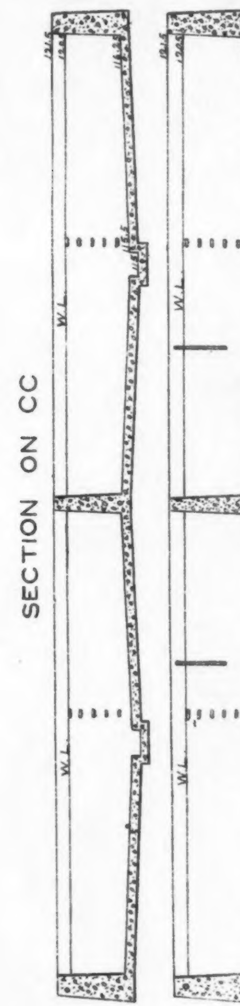


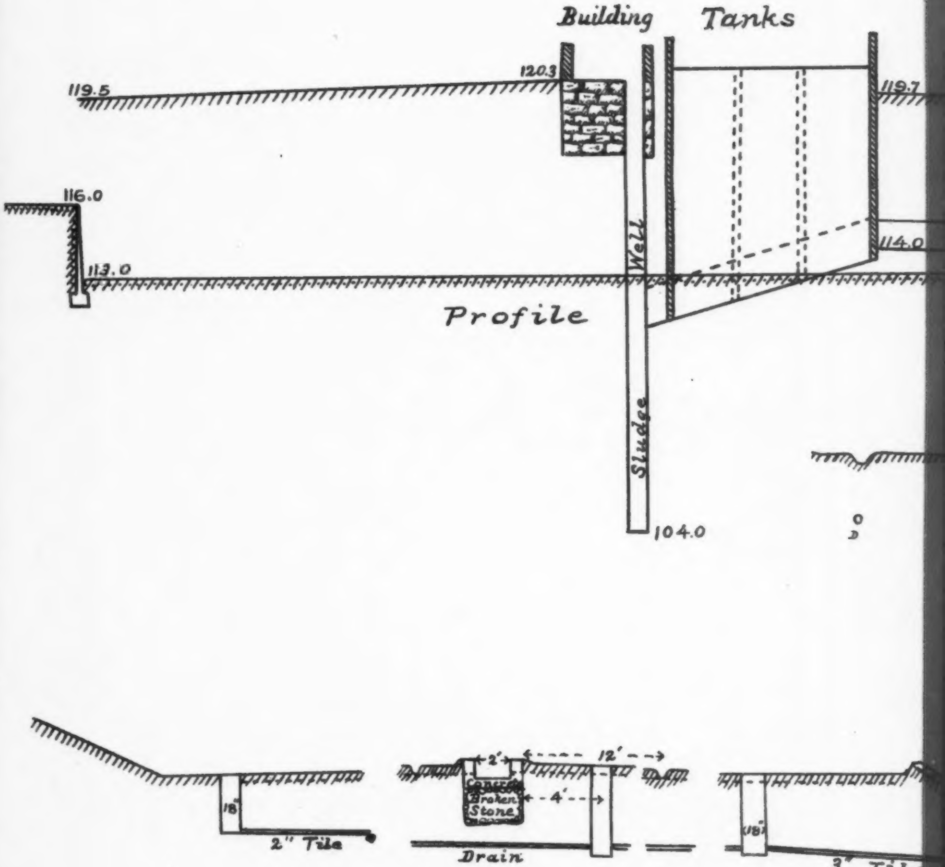


SECTION ON AA

BASSETT ON
INLAND SEWAGE DISPOSAL.



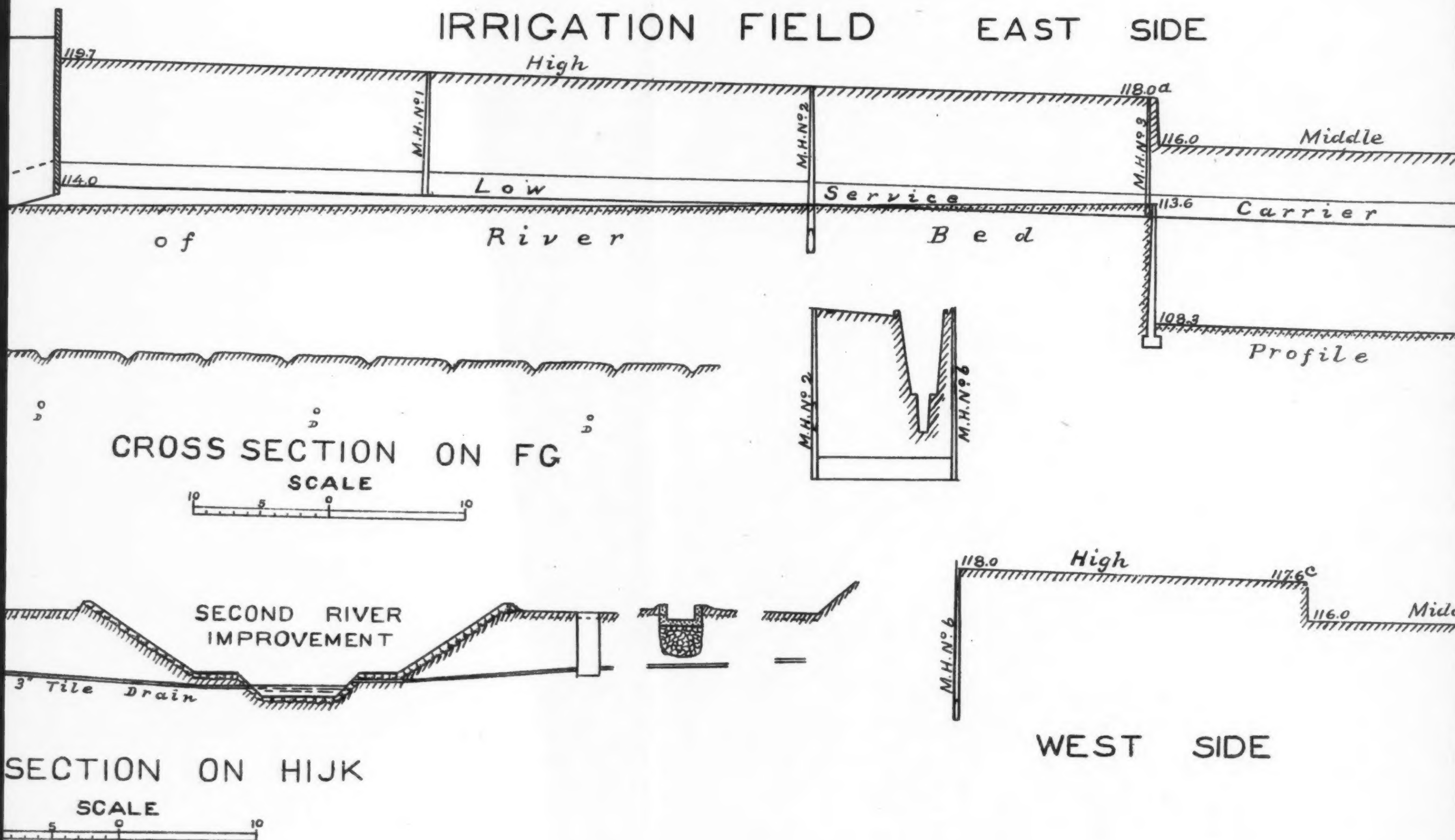




Building
Tanks
120.3
119.5
119.7
116.0
113.0
114.0
Well
Profile
Sludge
104.0
2'
12'
Concrete
Broken Stone
4'
18"
2" Tile
Drain
(18")
3" Tile

CROSS SECT

Profiles.



IRRIGATION FIELD EAST SIDE
119.7
High
M.H. No. 1
M.H. No. 2
M.H. No. 3
118.0 a
116.0
Middle
114.0
Low
Service
113.6
Carrier
of
River
Bed
108.3
Profile
CROSS SECTION ON FG
SCALE
10
5
0
10
M.H. No. 2
M.H. No. 6
SECOND RIVER
IMPROVEMENT
3" Tile Drain
SECTION ON HIJK
SCALE
5
0
10
118.0
High
117.6 c
116.0
Midd
M.H. No. 6
WEST SIDE

PLATE XXIII
TRANS. AM. SOC. CIV. ENG'RS.
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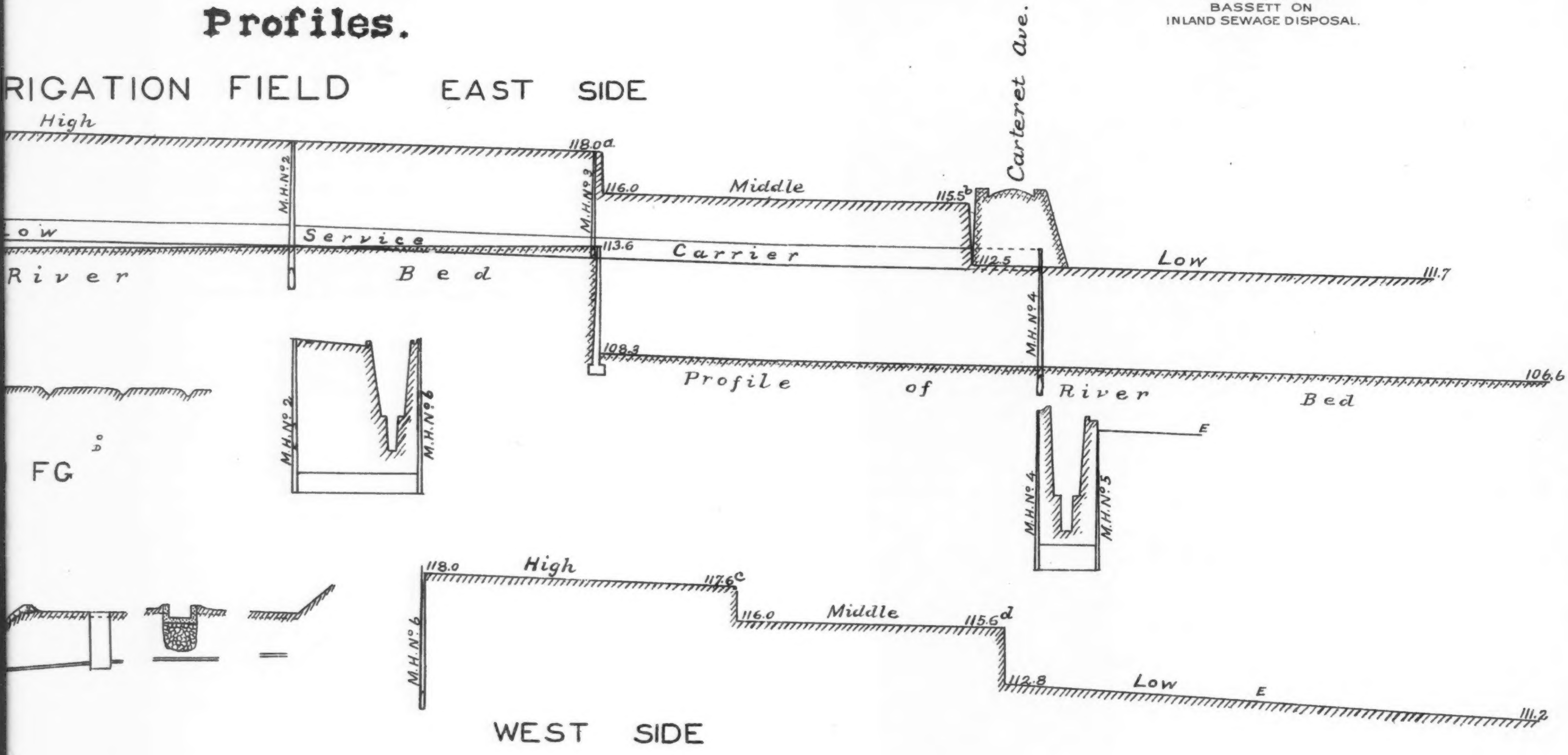
BASSETT ON
INLAND SEWAGE DISPOSAL.

Profiles.



pump), boiler, and a small office for records and tests (Plate XXII). On the second floor chemicals and materials are stored. The chemical mixers are cylindrical cast-iron vats, 4 feet in diameter, with inverted cone-shaped bottoms overlayed with a perforated plate. The desired amount of chemicals is placed on the plate, water is let into the tank, and air blown up through the bottom, causing violent agitation of the liquid and resulting in the rapid solution of the chemicals. With a known flow of sewage at a given time, it is determined how wide to open a slide-valve in the bottom of the tank after solution of the chemicals is secured, in order to add the desired number of grains per gallon of sewage.

The sewage is mainly of a domestic character and somewhat constant in its alkalinity. Not more than 3 grains of lime and 2 grains of sulphate of alumina are now added to each gallon of sewage by the authorities, although when the works were placed in operation, the author recommended the use of 8 grains of lime and 10 grains of sulphate of alumina per gallon of the sewage. The present result is a less efficient precipitation. A combination of chemical precipitation and land filtration in the works makes it possible to increase the work performed by the land, by reducing the efficiency of chemical treatment, and *vice versa.* The labor of purification now placed upon the grounds is greater than its equitable share as originally intended. Much better results could be secured by calling out the full efficiency of the chemical treatment. To relieve the filtration grounds, which have rather a retentive soil; several artificial filter beds of coke and gravel were constructed under my direction, and have been of material service. (See Plate XIX.)

Returning now to the precipitated matter or sludge in the tanks; after the supernatant water is drawn off through the swivel-arm into the low-service carrier, a valve gate is opened and the sludge drawn into the deeper sludge-well within the building. By forming a vacuum in a cast-iron receiver, which is connected by an iron pipe with the sludge-well, the sludge is drawn up into the receiver, milk of lime being drawn in at the same time, by a small pipe from the mixing tank in the chemical room. This lime prepares the sludge for pressing, cutting the slime so that the water separates more readily from the solids. A pressure of 100 pounds per square inch is secured in one of the other receivers, and, being connected with the receiver containing the sludge by an air transfer main and the proper valves opened, the sludge is forced into a Johnson filter press and pressed into moist, hard, portable cakes.

An analysis of fresh sludge directly from the press made September 11th, 1889, by Mr. Charles T. Pomeroy, of Newark, N. J., gave results as follows:

Nitrogen from organic matter	.326	per cent.
Total phosphoric acid	.459	"
Moisture	50.625	"

Using the 1889 trade values adopted by the New Jersey experiment station we have an estimated worth of $1.51 per ton, 2 000 pounds. This sludge quite rapidly lost its moisture on exposure to the air, until it contained 6.37 per cent. moisture. If dried at 100 degrees C. it would have an estimated value of $3.06 per ton, 2 000 pounds.

The machinery used in manipulating the sludge was constructed by S. H. Johnson & Co., Stratford, England, who have erected numerous plants in England. Their machinery is the subject of several patents, and no similar devices are manufactured in this country. Not all of the machinery furnished has been satisfactory. The combination vacuum and compression pump and the high pressure water pump gave considerable trouble, and have been replaced by a Clayton compressor (5′ x 6′ x 7′) and a Worthington duplex pump (6′ x 4′ x 6′).

The method of maintaining and securing the pressure in the "hydro-pneumatic receivers" to press the sludge is worthy of brief comment. There are three receivers in the system (see Plate XXV); let them be represented by *A*, *B* and *C*. Air passes out at the top, sludge at the bottom, water enters at the side near the bottom and exits at the bottom. All the receivers are empty, except of air. Water is pumped into *A* and *B* and the air transfer mains opened to transfer their air to *C*, raising the pressure to 45 pounds per square inch. The valve on the air main from *C* is closed. The water in *A* and *B* is drained back into a shallow tank in the floor of the building with which the pump suction is connected—air taking its place. Water is again forced into *A* and *B*, operating proper valves, and the pressure in *C* is raised to 75 pounds per square inch. A repetition of this process brings the pressure in *C* to 105 pounds per square inch. *A* and *B* are emptied as before and a vacuum in *A* created, the sludge suction pipe is opened, and *A* is filled with sludge. Water is now pumped into *B*, forcing its air into *C* and thence into *A*, to force the sludge into the press. When *B* is filled with water, *C* may be filled with air, and *A* with sludge and air. When a receiver is filled with water, a float valve at the top closes the outlet to the air transfer

PLATE XXIV.
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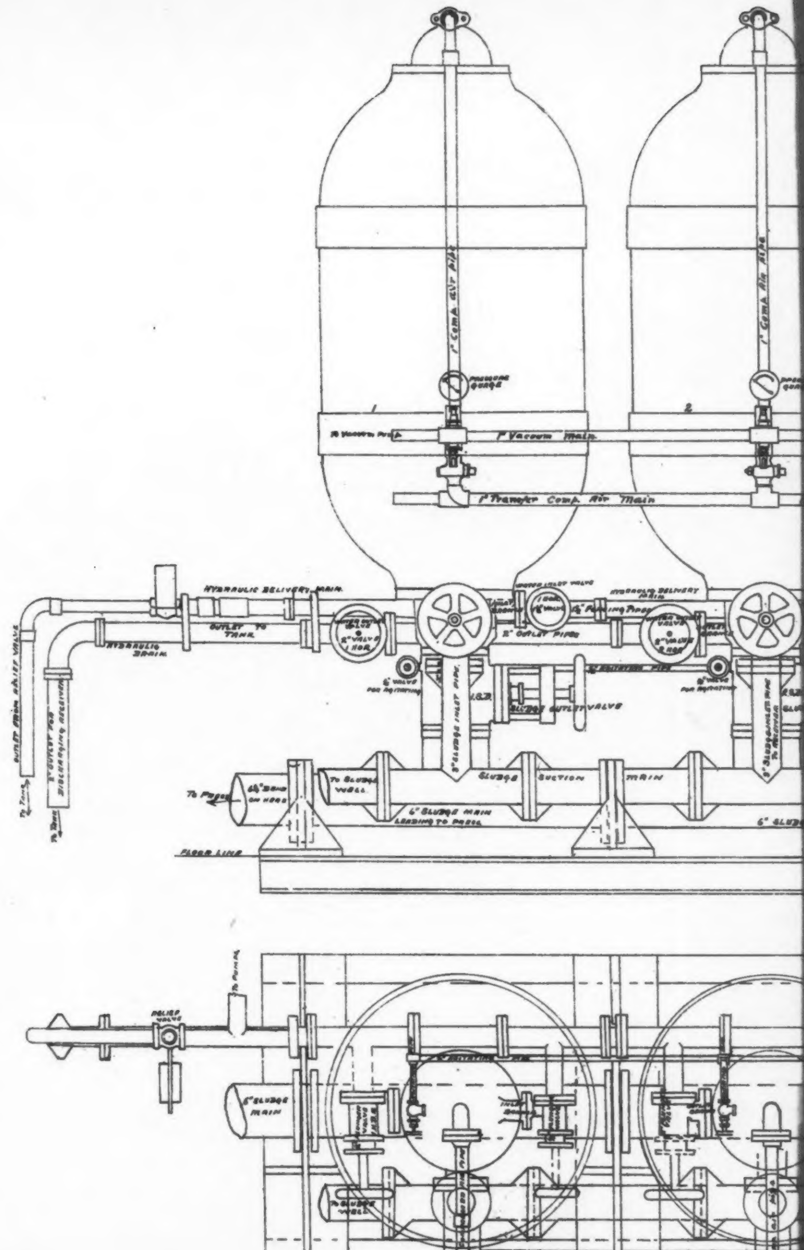
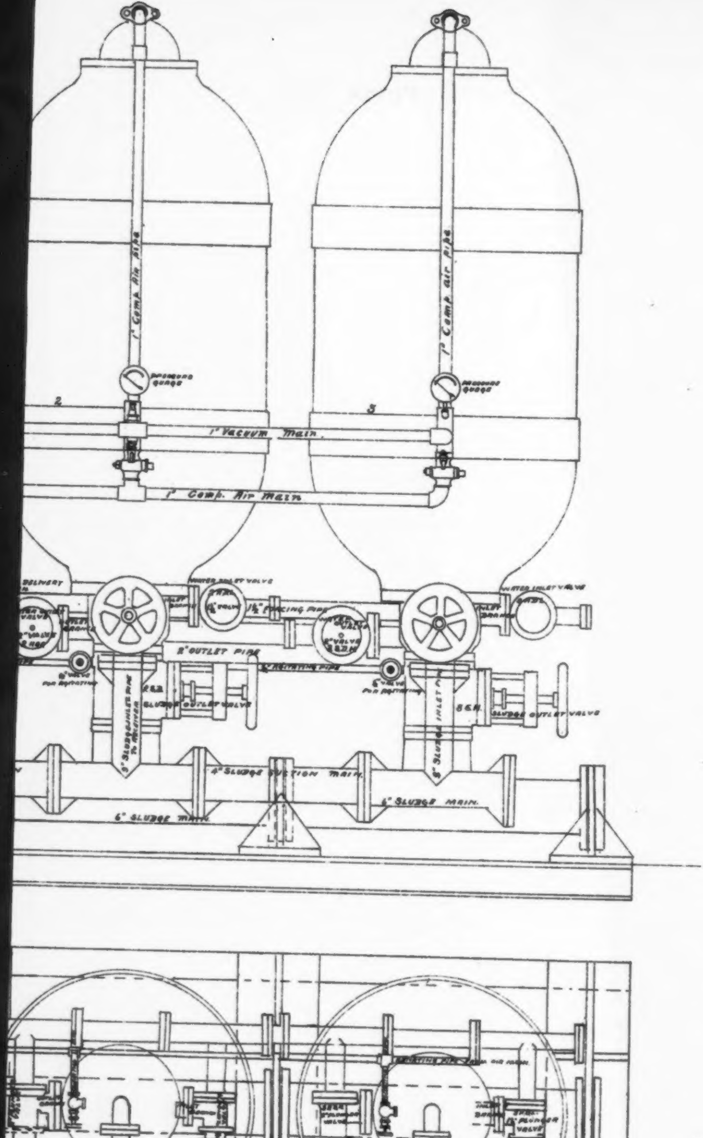
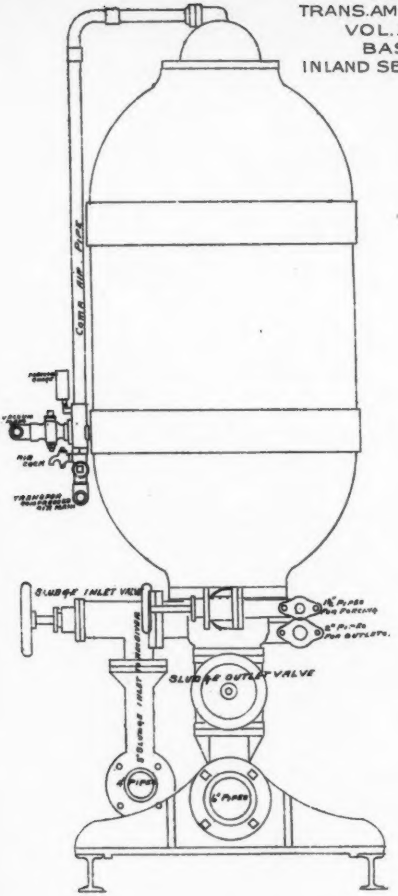


1" Comp. Air Pipe
Pressure Gauge
1" Vacuum Main
1" Transfer Comp. Air Main
Hydraulic Delivery Main
Outlet to Tank
Hydraulic Drain
2" Outlet Pipe
Sludge Outlet Valve
Outlet from Relief Valve
Sludge Suction Main
6" Sludge Main Leading to Press
To Sludge Well
Floor Line
Relief Valve
6" Sludge Main
Transfer Comp. Air Main
Vacuum Main


1" Comp. Air Pipe
Pressure Gauge
2
3
1" Vacuum Main
1" Comp. Air Main
Water Inlet Valve
1½" Forcing Pipe
8" Outlet Pipe
Sludge Outlet Valve
4" Sludge Suction Main
6" Sludge Main
4" Sludge Main



ARRANGEMENT OF SLUDGE FORCING RECEIVERS
FOR EAST ORANGE SEWAGE WORKS

C.PH.BASSETT
DES'NG CON'ST'G ENG.

main, and weighted valves on the force main lift and relieve the pressure on the pump.

The value of the process now first appears. B is emptied of water and filled with sludge, while at the same time water is being forced into C and by connection with A, forcing the remainder of its sludge into the press. Compressed air is thus never allowed to escape into the atmosphere. When C is filled with water, it is ready to be filled with sludge, while at the same time water is forced into A and the sludge of B forced in the press.

The filter press shown in Plate XXIV consists of thirty-six cast-iron cells, supported on a simple frame, with a central feed-passage into which the sludge is forced from the receivers. The cells are separated by canvas bags, and in the intercellular spaces the sludge remains, while the water is drained out through the canvas bags, into a trough on the rear of the press and returns to the tanks. On the end of the press is a capstan screw connected with a thrust block which presses the thirty-six cells of the press into close contact. It is the air pressure which separates the water from the sludge.

There is nothing offensive about these cakes when pressed dry; and if protected from water, after being taken from the press, they may be kept in bulk for weeks without nuisance. In the presence of heat and moisture they become more or less objectionable. The manurial value of the sludge cakes is slight. The small amount of precipitants used fails to retain the bulk of fertilizing matters in the sewage. At present between 8 000 and 10 000 people are contributing to the sewage and about 13 tons of sludge are taken out each week. Some of the sludge cake has been sold at fifty cents per load; but more has been given away among neighboring farmers, while a large amount has been carted away by the authorities for burial, when no other removal offered.

A committee of the "Town Improvement Society of East Orange," an association of more or less intelligent gentlemen, determined during the summer and fall of 1890 to investigate the operation of the works and their results. They secured and submitted to Professor A. R. Leeds, two samples of effluent water for examination and report. The results generally are better than results secured for the author's use from time to time, but Professor Leeds' report is given in full as published by the Committee, as independent testimony.

HOBOKEN, N. J., September 29th, 1890.

CERTIFICATE OF WATER ANALYSIS.

Sample No. 1. Taken from the flow as it emerges from the coke filter as the sewage leaves the tank.

(*a.*) From whom received, The Committee. No. 1447. When received, Sept. 8. Title of label, Sample No. 1. Source of Sample, E. Orange Disposal Works. Color, Turbid with white flocks. Taste, Not tried. Smell, Unpleasant, musty.

	Parts in 100 000.	Grains per gallon.
Free ammonia	0.087	0.051
Albuminoid ammonia	0.027	0.016
Oxygen reqired to oxidize organic matter	0.44	0.26
Nitrites	.00	.00
Nitrates	0.26	0.15
Chlorine	6.12	3.56
Total hardness	15.60	9.02
Permanent hardness	3.00	1.75
Temporary hardness	12.60	7.27
Total solids	29.60	17.26
Mineral matter	23.50	13.70
Organic and volatile matter	6.10	3.56
Other data, when required for judgment.		

INTERPRETATION OF RESULTS.

The results above stated indicate the presence of a large amount of unoxidized sewage. I should also suspect that this sample represents sewer water which has received the addition of some lime, taken at a point before the benefit of this treatment has been obtained. The effect of the addition of lime is to increase the temporary hardness (in the above analysis it is 12.6 parts per 100 000), after this lime has operated, by combining with the large amounts of carbolic acid contained in sewage, and the carbonate of lime resulting from this combination has precipitated, carrying with it much of the organic matter.

(Signed) ALBERT R. LEEDS, Ph.D.,

Professor of Chemistry, Stevens Institute of Technology.

NOTE.—The United States gallon is taken at 58 318 grains.

HOBOKEN, N. J., September 29th, 1890.

CERTIFICATE OF WATER ANALYSIS.

Sample No. 2. From the final effluent into the brook.

From whom received, The Committee. No. 1 448. When received, Septempter 8th. Title of label, Sample No. 2. Source of Sample, E. Orange Dis. Works. Color, White, clear. Taste, Not tasted. Smell, Not pleasant, slightly musty.

	Parts in 100 000.	Grains per gallon.
Free ammonia	0.02	0.012
Albuminoid ammonia	0.003	0.0017
Oxygen required to oxidize organic matter	0.40	0.233
Nitrites	.00	.00
Nitrates	0.38	0.22
Chlorine	4.00	2.33
Total hardness	20.00	11.63
Permanent hardness	12.50	7.23
Temporary hardness	8.50	4.40
Total solids	25.50	14.91
Mineral matter	22.00	12.83
Organic and volatile matter	3.50	2.08

Other data, when required for judgment.

It will be seen that the temporary hardness diminishes, and the benefit of the treatment becomes apparent. The second sample is strikingly different from No. 1. It shows an almost entire disappearance of the nitrogenous organic matter, the free ammonia being only $\frac{1}{10}$th and the albuminoid ammonia $\frac{1}{10}$th of the amounts present in sample No. 1. This disappearance is evidently due to oxidation, since the nitrates (which arise from the absorption of oxygen under the influence of nitrifying bacteria in the ground) are strikingly increased. I should suspect this sample to represent aerated sewage water which has passed through the ground. In its passage, its sewage impurities have been effectually removed, and the substances remaining are such as are found in country streams in their natural unpolluted condition.

Signed, ALBERT R. LEEDS, Ph.D.,

Professor of Chemistry, Stevens Institute of Technology.

NOTE.—The United States gallon is taken at 58 318 grains.

To make these results the more apparent, the following table is presented:

		Free Ammonia.	Albuminoid Ammonia.	Oxygen required to oxidize Organic Matter.	Nitrites.	Nitrates.	Chlorine.	Total Hardness.	Permanent Hardness.	Temporary Hardness.	Total Solids.	Mineral Matter.	Organic and Volatile Matter.
Raw sewage......	Parts in 100 000....	1.0 to 1.5	.30 to .70	....	......	......	5 to 10	.....	......	8 to 12	40 to 108	18 to 91	7 to 22
Effluent from.....	Parts in 100 000....	0.087	0.027	0.44	0.0	0.26	6.12	15.60	3.00	12.60	29.60	23.50	6.10
Coke filter........	Grains per Gallon ...	0.051	0.016	0.26	0.0	0.15	3.56	9.02	1.75	7.27	17.26	13.70	3.56
Final effluent	Parts in 100 000....	0.02	0.003	0.40	0.0	0.38	4.00	20.00	12.50	8.50	25.50	22.00	3.50
	Grains per Gallon....	0.012	0.0017	0.233	0.0	0.22	2.33	11.60	7.23	4.40	14.91	12.83	2.08

In a letter of later date explanatory of these analyses, Professor Leeds stated:

"I should certainly feel far less sense of danger in drinking the sewage effluent, as represented by the samples sent from the East Orange

Sewage Disposal Works, than in drinking the water of the Passaic River, as pumped at Belleville and supplied to the inhabitants of Jersey City. Your effluent (September 8th, 1890) contained 0.0017 grains of albuminoid ammonia per gallon; the Passaic River, at Belleville, on the date of my last analysis (June 14th, 1890) contained 0.01 grains per gallon. In other words, taking the albuminoid ammonia as the measure of sewage contamination, the Jersey City water contains six times more sewage than the effluent waters from your works.

" As chemist for the Jersey City Board of Public Works, from 1881 to 1886, I found very many samples of the Passaic water even worse than the above.

" I regard the performance of your works and the character of the effluent as satisfactory from both the practical and sanitary standpoints."

The total cost of the works to January 1, 1891 (including about 4 miles of extensions constructed since my connection with the work), is given as follows:

Chargeable against sewerage system (29 miles)..	$322 020 64
Disposal works plant........................	75 098 60
Disposal works land (including 4 acres not used)	20 749 20
Total...........................	$417 868 44

The cost of operating the works has been—

July, 1888, to March, 1889............	$562 00	per month.
March, 1889, to March, 1890..........	746 00	"
March, 1890, to January, 1891.........	881 00	"

The average daily flow of sewage reaching the disposal works is approximately 1 300 000 gallons, or an average of about 90 000 gallons per acre. The need of the coke filters is therefore apparent. This daily flow may be approximately divided as follows:

Ground water from 25 miles, constructed under my direction..................	550 000	gallons.
Ground water from 4 miles, constructed since June, 1888..................	100 000	"
Flush tank flow.........................	30 000	"
House sewage flow.......................	620 000	"

Summarizing the processes of purification operating on the sewage in its passage through the works, we find subsidence, coagulation and precipitation in the tanks, mechanical filtration, aeration and nitrification in the soil. These principles include all that were practically available at the time the works were designed. More efficient precipitation might be secured, better land and more of it would be greatly appreciated, better business methods about the works and the disposal of its products are to be hoped for, and yet, under all the conditions involved, the plant is a success. It is doing its work under unfavorable circumstances, and doing it to the satisfaction of the East Orange authorities and its neighbors.

DISCUSSION.

Mr. RUDOLPH HERING, Director Am. Soc. C. E. (to Mr. Bassett).—You gave the quantity of ground-water entering the sewers. Was it determined before any sewage was turned into the system?

Mr. BASSETT.—My original determination was made before any sewage was turned in, by gauging the outflow sewer and also by noting the length of time required to fill the tanks whose capacity was known.

Mr. HERING.—I remember asking you to do this, because later it would have been impossible. The present cost of maintenance you say is over $10 000 a year.

Mr. BASSETT.—It is so reported by the township authorities.

Mr. HERING.—And the population contributing sewage to the works is between eight and ten thousand, making the cost of treating the sewage over $1 per head per annum. This expense is greater than it ought to be for the process used. I believe the excess is attributable to the fact that a large quantity of ground-water must be taken care of by the tanks and the filtration areas, instead of being discharged directly into the Second River. What did you say was the proportion of sewage to ground-water?

Mr. BASSETT.—Pretty nearly equally divided.

Mr. HERING.—That, I think, explains the large expense. When the work was projected it was not apparent that so much ground-water was to be encountered, and great care was taken during construction to keep it out of the sewers. In order to prevent the ground-water from entering them and therefore getting into the tanks, the only practicable remedy is a system of under-drains independent of the sewers. I am not as sanguine as some are, regarding the possibility of obtaining a high degree of purity by any method of precipitation at a reasonable expense. Experience has shown both here and in Europe that it is not practicable

to so purify sewage by any known chemicals that the effluent water will not again putrefy. The value of precipitation must be sought in the removal of a large portion of putrescible matter, particularly what is suspended in the sewage. This leaves the effluent in a condition to be purified by filtration through a smaller area of land, or by dilution in a smaller body of water than would otherwise be possible. In other words, where insufficient land of a suitable character (sand) is available for the filtration of a given amount of crude sewage, such as was the case in East Orange, or, where the stream is too small to properly dilute the crude sewage, as in Worcester, Mass.—there a prior sedimentation or precipitation of some kind, aids us in finally getting a high degree of purity. The complete purification of sewage seems to be accomplished only by bacterial action, which, in the presence of oxygen, converts the organic matter into inorganic compounds. Oxygen, warmth, and time are essentials in the process. Slow and intermittent filtration through sand, or dilution in water having a large quantity of oxygen in solution, therefore, accomplish a complete purification of sewage in due time.

Experiments upon the chemical precipitation of sewage were made by the Massachusetts State Board of Health in December, 1889,* with lime and with salts of aluminum and iron, in order to discover their comparative value for the purpose. It was found that a certain definite amount of lime gave as good and better results than either more or less, and that this amount was equal to that which neutralized the carbonic acid in the sewage, or nearly so. It was further found that the more alum, copperas, or sulphate were used the better was the result; and that, while alum and ferric sulphate usually require no lime to perfect the precipitation, copperas requires a definite amount to do so. Copperas alone produced no better results than simple sedimentation. To accomplish precipitation by it, it is necessary to have more than enough lime to combine with the excess of carbonic acid over the amount to form bicarbonates and to combine with the sulphuric acid of the copperas. The relative value of the above chemicals will be shown in the following table, assuming the percentage of albuminoid ammonia removed as representing the organic matter, the cost of the chemicals being the same in each case.

SOLUBLE ORGANIC MATTER REMOVED IN ADDITION TO ALL SUSPENDED MATTER.

	Per Cent.
Aluminum sulphate, material costing thirty cents per inhabitant per annum	20
Lime, material costing thirty cents per inhabitant per annum	22
Lime and copperas, material costing thirty cents per inhabitant per annum	29
Ferric sulphate, material costing thirty cents per inhabitant per annum	32

* Part II on Water Supply and Sewerage, 1890.

From the above experiments it appears therefore that at the same cost, ferric sulphate gives the best, and alum sulphate the least good results; also that the use of lime and copperas together, in the proper proportion, gives better results than either lime or copperas alone. It was found by the same series of experiments that both aluminum and ferric sulphate gave about as good results without lime as with it. The precipitation would be a little more rapid but hardly enough so to compensate for the extra cost. As lime and aluminum sulphate are the materials used at East Orange, it would seem that the use of copperas instead of alum, or still better ferric sulphate alone, would at a reduced cost give equally good results as those now obtained. If, still further, the groundwater could be excluded, the annual cost of the purification would be very materially lessened.

Alfred P. Boller, Mem. Am. Soc. C. E.—Mr. Bassett having referred to a committee of the Town Improvement Society, of East Orange, from whose report he does them the honor to quote liberally, as chairman of that committee, it is proper for me to add a few words to his interesting paper. This committee was the outgrowth of very grave charges against the efficiency of the Disposal Works, which implied the utter failure of the precipitation process, and that of the filtration of the effluent; resulting in the practical discharge of the sewage matter into the brook, substantially as it came from the outlet sewer. The result of the committee's findings completely disposed of these charges, the improvement being largely due to the betterments and administration of Mr. James O'Neill, the Township engineer in charge. Previous to his administration much trouble had arisen from the inefficient English machinery (referred to by Mr. Bassett); the failure to have an independent pump to return the thin sewage water from the sludge-well back to the settling tanks, instead of trying to pass it through the receivers only adapted to care for the thick and pasty sludge; and the failure of the filtration grounds which did not filter, due to the retentive nature of the clay soil, with the drain pipes 4 to 6 feet under the surface. These difficulties have been overcome by putting in new compression and water pumps, as stated by Mr. Bassett, and the addition of a pulsometer pump, to relieve the sludge-well of excessive water, and the introduction of coke filters in the form of beds of solid coke, some 50 feet long, 6 feet wide and 5 feet deep. Such beds are independent of all weather, but must be renewed when clogged.

To carry the sewage over the ridge from the low ground in the eastern portion of the township, an automatic gas engine had been located in a vault under Park avenue, near Grove street, which quickly proved to be utterly incapable of performing the work required of it. Much trouble, as can be imagined, grew out of this failure, but a new and efficient pumping plant, differently located, has been established and in

operation since July, 1890, correcting the evils this district suffered from.

The extent of leakage from ground-water (computed at about one-half the amount of sewage water treated) was obtained from gaugings at different times and hours in the outlet sewer as it enters the disposal works, compared with the number of times in twenty-four hours the settling tanks were filled. From this was deducted the legitimate work of the sewer system on the basis of the number of house connections and the water furnished by the Orange Water Company, the difference being ground water as near as it is practicable to obtain it. As Mr. Bassett truly says, the character of the excavations rendered tight work with vitrified pipes and cement joints absolutely impossible, and it is purely a question of individual judgment whether the leakage is in excess of what it fairly should be.

As the works are now operated, they can be called a success, and I cannot close my comments better than by quoting the opinion of the Committee at the close of their report: "The Committee are of the opinion that ultimately it will be found economical to dispose of the sewage by gravity to tide-water, which may become practicable through united action with the adjacent towns. The maintenance of a gravity system is practically nothing, while that of any local system, like that of East Orange, is a continuing and increasing annual charge. The town can well afford to capitalize such a charge and contribute accordingly to a gravity outlet, while the sewerage account could be reduced by the sale of the present filtration grounds. East Orange, however, can await with perfect equanimity the developments of the future, taking advantage of whatever may prove to be to its interests when the time arrives. Its present system of sewage disposal is a success and working satisfactorily, and can be made to serve the needs of a much larger population than the township now contains."

George W. Rafter, M. Am. Soc. C. E.—Mr. Bassett's paper is somewhat disappointing. Considering the lack of literature of sewage disposal in the publications of the Society his title led to the hope that the whole subject of sewage disposal in localities away from tide-water would receive a thorough overhauling at his hands. His preliminary discussion is, of course, in the line of such overhauling and will be of interest to those who have not given the subject special consideration. There are, however, a few debatable points raised, as for instance, Mr. Bassett expresses the belief that in some form of chemical precipitation lies the great hope for sewage purification in compactly populated districts. This may possibly be true, but in the present state of knowledge of sewage purification it may, I think, be considered doubtful, and I venture to submit a short discussion, not in the way indeed of attempting to prove absolutely the contrary proposition, but rather for the pur-

pose of showing that no sweeping rules of procedure, governing every case arising can be laid down.

In studying sewage purification historically it is found that the weight of opinion has vibrated pendulum-like between the different methods, and for the last few years chemical methods have been somewhat in the ascendant. In the case of the town of East Orange, it appears clearly necessary to manufacture potable water from the sewage, and the inadequacy of the chemical method by itself to do this, is indicated by the character of the design proposed by Mr. Bassett for the sewage purification works at that place. The chemical analyses of the effluents made by Dr. Leeds are of interest as showing the relative value of the chemical methods as compared with purification by land. On the question of relative cost I shall invite attention directly.

From Dr. Leeds' analyses it appears that at East Orange the effluent from the chemical purification process is still practically unpurified sewage. This is emphasized by the remark of Dr. Leeds, to the effect in his interpetation of results, that he suspects "that this sample represents sewer water which has received the addition of some lime, taken at a point before the benefit of this treatment has been obtained." This is especially significant as to the value of the chemical treatment when we consider that the sample on which Dr. Leeds makes this report was, according to the record, "taken from the flow as it emerges from the coke filter as the sewage leaves the tank." That is to say, if I understand the matter correctly from Mr. Bassett's written description, this sample was taken after the chemical treatment and after the sewage had further flowed through a coke filter. The second sample was from the final effluent after the land purification, and represents water, against the potable qualities of which no valid objection could be urged from the chemical point of view. Dr. Leeds, however, records the smell as "not pleasant, slightly musty," a condition possibly superinduced by the character of the soil through which the filtration takes place. Mr. Bassett remarks that these two analyses of effluents as made by Dr. Leeds are generally better than the results secured for his own use from time to time. If Mr. Bassett has anything like a complete series of analyses of these effluents it would be of considerable value, as assisting in settling this question of the relative efficiency of the chemical and land purification methods, if he would introduce them in his final discussion.

It is now well understood that a single series of analyses cannot be considered as indicating anything more than the condition of things at the time of taking, but if those made by Dr. Leeds are fair averages, we may conclude that the analyses now before us are a striking illustration of the truth of some remarks on the insufficiency of chemical methods made by Dr. Carl Pfeiffer of Wiesbaden in 1888, namely:

"For some incomprehensible reason, this entirely impracticable and,

as regards its results, most unsatisfactory mode of treatment has, during the last few years, grown much in favor, and the author is of the opinion that the time has arrived when a strenuous opposition should be offered to these so-called clarification processes, and, when in the interests of municipal authorities, a warning should go forth against the excessive cost of the chemical systems of treatment as compared with the good they can effect."

Where conditions similar to those at East Orange exist, where it is absolutely necessary that the purified effluent pass into streams used for water supply below the sewage outfall, the prime object of sewage purification is and must always continue to be the removal not only of suspended and dissolved organic substances but, what is of far greater importance, of disease germs. The analyses submitted by Mr. Bassett show that the chemical treatment had removed only a portion of the organic matter, and we have absolutely no guarantee under these circumstances that any disease germs have been removed at all. Probably, however, the coagulation and attendant precipitation superinduced by the addition of the chemical reagents may have carried down into the sludge some of the bacterial germs of disease if any were present, but that anything like all the germs are ever removed in this way we have as yet no reason for assuming.

In view of the foregoing, let us examine (1) as to whether the work actually accomplished by the chemical treatment is really worth what it costs; and (2) whether in the light of present information the same work could not be better accomplished by other methods.

Mr. Bassett states that at the present time lime is added at the rate of not more than 3 grains per gallon of sewage and sulphate of alumina at the rate of 2 grains per gallon, and that the daily flow of sewage is at the rate of approximately 1 300 000 gallons. For purposes of the discussion I will assume that lime of the proper quality is delivered at the works in the bins ready for use at $8 per net ton, and sulphate of alumina at $22 per net ton. The yearly cost for these chemicals becomes then:

Lime, 101.6 net tons, at $8....................	$812 80
Sulphate alumina, 67.8 net tons, at $22..........	1 471 60
Annual cost of chemicals..................	$2 284 40

With the assumed prices the foregoing represents the annual cost of chemicals under the present system of operation, while the total cost of operation is now, as per statement, $881 per month or $10 572 per year. It would assist greatly in studying the question of efficiency of these disposal works, if Mr. Bassett would give in the final discussion the detail, as for instance, cost of fuel, oil and waste, chemicals, attendance for the chemical treatment purely, the same for the land purification process, together with the cost of renewals and repairs for each.

The cost of operating the Johnson Filter Press would also be an interesting and valuable item of information, especially since this is, so far as I know, the only Johnson press in operation in this country. As the matter now stands the cost of the operation of those works appears to be more than $1 per head per annum for the population actully served, and this must in our present light be considered somewhat high, even in a case where the conditions require the manufacturing of potable water.

Taking my assumed annual cost of chemicals, it appears that the other items of expense are as a whole ($10 572 00 — $2 284 40) = $8 287 60. Mr. Bassett, however, tells us that his original recommendation was 8 grains of lime and 10 grains of sulphate of alumina per gallon of sewage. Let us consider the total annual expense if this recommendation should be carried out:

Lime, 271.2 net tons, at $8.................. =	$2 168 80
Sulphate alumina, 338.9 net tons, at $22...... =	7 455 80
Annual cost of chemicals.................	$9 625 60

Adding this amount to the actual cost at the present time, exclusive of chemicals, and we have ($9 624 60 + $8 287 60) = $17 912 20, a sum representing approximately $1 80 per head of population per annum.

Mr. Bassett is apparently under the impression that the greater the use of chemicals the greater will be the degree of purification attained by chemical treatment, but experience shows that beyond a certain limit no material advantage accrues from the use of additional chemicals. This conclusion is supported by the result of trials at the Mystic Valley Sewage Purification Works as detailed by Wilbur F. Learned, Member of the Boston Society of Civil Engineers, in a paper read February 15th, 1888; from which it appears that the addition of sulphate of alumina at the rate of one-half ton per 1 000 000 gallons precipitates at the Mystic Valley Works 25 per cent. of the total matter of the sewage, while two tons per 1 000 000 gallons precipitates generally only about 30 to 32 per cent. of the total matter. In this case but one reagent is used, and it is only fair to state that better results are obtained by the use of two reagents than one. My only object in citing this experience at the Mystic Valley Works is to illustrate the principle that beyond certain limits the addition of chemicals has little effect in the way of assisting the purification. Moreover, it may be noted that the amount of chemical reagent to be used will depend to some extent upon the amount of dilution of the sewage. Dr. Lidy states that for a successful lime treatment the quantity of lime added should be at least 10 grains per gallon, to a sewage that does not exceed 30 gallons per head of the population, but probably this amount was intended for sewage, the watery

portion of which is comparatively hard. Where two reagents are used the amount of each is much less than with only one, and for the lime and sulphate of alumina process 5 grains of each may be taken as a maximum for sewage of medium strength.

The precipitating effect of lime on sewage is due to two causes: (1) the combination of some of the lime with free and partially combined carbon dioxide, forming an insoluble carbonate of lime; and (2) to a further combination of an additional part of the lime with a portion of the organic matters in solution. The insoluble substances so formed subside, carrying with them the major portion of the suspended matters in the sewage, and sinking to the bottom they form sludge.

Sulphate of alumina exercises a precipitating effect by virtue of a combination of the sulphuric acid with lime and other bases in the sewage, whilst alumina hydrate forming a flocculent precipitate entangles and carries down the suspended organic matters. The most of the authorities now recommend the lime and sulphate of alumina treatment, the proportions in which they are used being such as to yield as nearly as possible a neutral effluent. It is evident, therefore, that no hard and fast rule as to the amount of reagent to be added can be laid down. Theoretically, frequent tests should be made of the nature of the sewage as delivered at the disposal works, and the chemical treatment adapted to the varying conditions of the flow. Practically, also, this has been found to be the best method of procedure, and it is stated that at Worcester such tests are made, and the application of chemicals gauged in accordance with the results thereof, as often as once every half hour.

At East Orange, with a population of 10 000, and a flow of sewage of 1 300 000 gallons in twenty-four hours, we have a daily flow per capita of 130 gallons. With such a dilution of the sewage, I question somewhat Mr. Bassett's conclusion, that very material improvement of the effluent from the chemical treatment will be obtained by the use of the quantities originally recommended, over what is now obtained. Certainly the slight gain in purity of effluent would not be commensurate with the expense. My reasons for this opinion I indicate in the foregoing. There are, moreover, good grounds for believing that the purification of the quantity of sewage treated at East Orange could be efficiently accomplished, even on the limited area there available, by the process of intermittent filtration, at materially less expense. The experiments of the Massachusetts State Board of Health have pointed out to us the rational method of procedure, and much of the information available upon the subject of intermittent filtration when Mr. Bassett designed these works is obsolete, by reason of the new views which we derive from the Massachusetts experiments. I will not take time in discussing at length the results which have been obtained in these experiments. The large volume containing the record of them is now in process of distribution, and it is sufficient for present purposes to say, that with a

sewage as dilute as that at East Orange, there is no reason to doubt but that intermittent filtration areas can be constructed, capable of taking care of and efficiently purifying for all time to come, from 75 000 to 100 000 gallons of sewage per acre per day; and taking 75 000 gallons per acre per day as the basis of an estimate, we find that 17 acres would be the area required. I understand, however, that 19 acres were actually purchased at East Orange, and assuming that this is a nearly level tract, we will, for purposes of comparison, estimate the cost of preparing this area for high grade intermittent filtration, in accordance with the Massachusetts views. In this estimate I take the price of sand at $1 per cubic yard, in place in the work, the assumption being that for the quantity required to make a depth of 3 feet over the whole area, this price would be ample, even under somewhat unfavorable conditions, as for instance, long haul and the necessity for laying some temporary track, in order to transport sand from the bank by railway transportation.

A filtration field of this character will be prepared, under the conditions which obtain at East Orange, where the sewage can be easily delivered upon it, with no other assistance than gravity, by first leveling the natural surface in beds, substantially as has been actually done by Mr. Bassett. After such leveling the under drains will be laid, the trenches refilled, and selected sand of the proper quality filled in upon the previously leveled and prepared surface, the material moved in the process of grading and leveling having been used to form the necessary embankments at the sides. The designs of the distribution carriers and necessary arrangements for straining out coarse material, is a matter of simple detail, which need not be considered here.

The estimated cost of such a field, sufficient for the work to be done at the present time at East Orange, may be taken as a matter of illustration, subject to the limitations indicated, as follows:

19 acres of land, as per paper		$20 749 20
19 acres leveled and graded, at $300		5 700 00
19 acres drained, at $250		4 750 00
19 acres furnished with sand 3 feet deep, at $4 800		91 200 00
Distribution carriers and arrangements for straining out coarse material		10 000 00
Barns, sheds, teams, wagons, tools, etc		2 000 00
Contingent expense		15 600 80
Amount		$150 000 00
Annual cost of operation :		
1 foreman, at $75 per month	$900 00	
6 laborers, at $35 per month	2 520 00	
Keeping teams, repairs of tools, etc	1 000 00	
Amount	$4 420 00	
$4 420 capitalized at 4 per cent		110 500 00
Total capitalization		$260 500 00

Taking the cost of the purification works as actually constructed, as stated by Mr. Bassett, and we have the following:

Disposal works, plant	$75 098 60
Disposal works, land	20 749 20
Total cost of disposal works	$95 847 80
$10 572, the present cost of operation, capitalized at 4 per cent., gives	264 300 00
Total capitalization	$360 147 80

From which it appears that something like $100 000 capitalized investment would be saved at East Orange by the method of intermittent filtration, the comparison being made with the actual cost of the chemical process as carried on at the present time. If, however, we assume for the sake of the argument, that the chemical treatment originally recommended be applied, we find that the difference in the total capitalization in favor of high grade intermittent filtration will be $283 152 80.

I have gone somewhat more into the detail of this matter than I intended when I started out, and the only excuse I can offer is that it is a subject of absorbing interest to those who have studied it sufficiently to become familiar with the detail. I hope the introduction of such detail in the discussion is not wearisome to the Society. There is, however, one other point growing out of the Massachusetts views as to intermittent filtration, namely, that the preparation of these high grade filter areas in localities where the proper material is available, by reason of reducing very materially the number of acres required to filter the sewage of a given population, has a most marked effect upon the cost of maintaining and operating the intermittent filtration areas. Obviously much less labor will be required to operate 20 acres than would be required for 60 acres, and we therefore must conclude that the foreign experience in the cost of operating intermittent filtration areas can have in the present understanding of things, relative value only. The use of such data, without modification, will lead to erroneous results, and estimates of the cost of operation which do not take into account the new views are behind the times.

Enough has been said in the foregoing to indicate that there is a good deal of doubt as to chemical treatment being the great hope for sewage purification in compactly populated districts. Indeed, where a high degree of purity of effluent is required it can be usually attained at less cost by intermittent filtration alone than it can be by the combination treatment which Mr. Bassett has used at East Orange. At the same time, I do not wish to be understood as pinning my faith exclusively to intermittent filtration. In sewage purification, of all things, the engineer must look at the question from every side.

Mr. Bassett's closing remark that better business methods about the works are to be hoped for, leads to the inference that he has probably experienced in connection with this design some of the difficulties surrounding the engineer striving to attain an ideal in the prosecution of municipal work. Just how difficult this is, under the present universal system of political management, is well understood by all honorable municipal engineers. Such criticism as is indulged in as to the apparently high cost of operation of these works is fully subject to modification, due to the probable political co-efficient, and I trust that Mr. Bassett will understand, therefore, that my criticisms are made merely in the hope of eliciting the truth.

C. Ph. Bassett, M. Am. Soc. C. E.—In closing the discussion I desire to state that I have had no official connection with the East Orange works since their completion and delivery to the town in the summer of 1888. Management of the plant has not been under my control, and my information regarding expenses is necessarily obtained from officials or town records. Many of the expenses chargeable against the works are properly chargeable to betterments and experiments, and not to maintenance, but I have been unable to fully separate these items. This is seriously regretted, for a much more gratifying showing could be made were items of maintenance proper placed alone.*

The remarks of Mr. Boller apparently convey the impression that the coke filters at the works are recent additions. They were in fact built originally under my direction in 1886 and 1887. They have, however, been renewed. It might also appear from the temper of Mr. Boller's reference to the small pumping station in the eastern section of the town, that some censure was due the original location and design. The station was constructed as a vault beneath Park avenue (as indicated on Plate VII), because the territory to the north was entirely undeveloped and could not be opened by an outfall sewer without troublesome litigation. The committee in charge accepted the location as an expedient until the property was opened at the will of the owners. Steam could not be used in a vault beneath this important throughfare, and a

* I have ascertained from Superintendent O'Neill that the expenses of operation at the disposal works at the present time are approximately as follows:

Engineer and laborers at building, coal and water, oil and waste	$300	per month.
Chemicals, including lime	200	"
Manager and two helpers on grounds	155	"
Removal of sludge	70	"
Total	$725	"

There are probably 15 000 people contributing to the sewage at the present time. The annual per capita cost of maintenance, therefore, exclusive of interest charges is about 60 cents.

It is probable that this *per capita* cost will reduce as experience and sewage volume increase.

Baldwin gas engine geared to two small Worthington pumps was used. The phenomenal growth of the district served by this machinery, immediately following the construction of the sewers, made its usefulness shortlived; but the growth caused streets to be opened in the lower district, and made possible the construction of the pumping station practically where originally located by me.

Kindly remember that the works described in this article were designed in 1886, and my original paper was presented at the Cresson Convention in June, 1890, and now appears substantially as then read. The works were the first in this country to apply a systematic chemical treatment of sewage. European experience alone guided their design. The preliminary statements of the paper were intended to outline what had then been done in this country in sewage purification. Since June, 1890, much work of experimenting and actual construction has been accomplished. Members in their discussion have had access to this recent information, and naturally, have been surprised that it did not appear in my paper now published. I personally much regret that the paper did not appear soon after it was first presented. It may be mentioned that the delay was not entirely mine. It is not my intention in this discussion to bring the subject fully up to date; that must be left for later and more comprehensive papers. But I desire to add a few words along the lines originally considered, and reply to the inferences drawn by members in their discussion from recently acquired information which reflect on the principles of the works as designed in 1886.

It is to be regretted that provision for underdrainage in parts of the East Orange system were not made. The conditions conspiring against this result have been outlined, but it appears desirable to emphasize a statement in my original paper, that the large ground water flow entered the system mainly from about 3 200 feet of brick sewer. At the outlet sewer (1 700 feet long) ground water was not expected in large volumes, and the work began in a dry trench; an under drain was not therefore laid. This work later penetrated a very wet quicksand. At the other point (1 500 foot tunnel), objection was raised by the committee in charge to the expense of underdrainage into a lateral valley. Had the water been excluded from these two pieces of brick work, the ground water flow from the 25 miles of sewer would not have exceeded 400 000 gallons per day. The expense of operating the works would then have been much reduced. The quantity of ground water flow as I have given it in the original paper was ascertained by me before any house connections were made. Mr. Boller's remarks on this matter I fancy refer to the calculations of the committee which appear to accord well with my own gaugings. For the sewerage of Orange, N. J., immediately adjoining East Orange, and in the same class of construction, I am at the

present time securing effective underdrainage in what is called "liver" and running sand, and securing practically dry sewers.

I am at one with Mr. Hering in his statements regarding the present limitations of chemical precipitation, with one exception. He appears to me to underestimate the value of this process. He states that the experience of Europe and this country has shown that it is not possible with any known chemicals to so purify sewage that it will not again putrefy. Recent reports, now in hand from Europe, are not entirely in harmony with this statement. Sir Henry Roscoe, in his report on the effluent from the Webster electrical process, states: "I have not observed in any of the unfiltered effluents from this process, which I have examined, any signs of putrefaction, but on the contrary, a tendency to oxidize." Alfred E. Fletcher, Esq., F.C.S., Inspector under the Rivers Pollution Prevention Act in Scotland, states: "The results of my examination of this process convince me of its efficiency in preventing putrefaction of the effluent." J. Carter Ball, F.I.C., etc., states in his report on the experiments at Salford, England: "In comparing the two systems the electrical and the polarite, it is rather difficult to state which yields the better effluent in regard to liability to putrefaction. Many of the effluents have been kept from both processes for weeks and months in a warm place in open vessels, and at the end of that time there was not the faintest odor of decomposition, and in most cases the water is clear." Prof. E. Frankland, D.C.L., comments in a similar vein upon the process at Acton, England.

I have entered thus fully on this matter since the expression of my belief that in some form of "chemical precipitation lies the great hope of sewage disposal in compactly populated districts" has occasioned some criticism. This belief did not (in 1890) grow out of the superior results then obtained from chemical processes, but from a consciousness of the deep-seated popular aversion to a sewage farm near habitations, and a realization that in many districts, owing to lack of unoccupied land, land disposal is a practical impossibility. If chemical processes can be sufficiently perfected so that a large percentage of the dissolved—and practically all the suspended matters—can be economically removed, we will see less effort made to unite large thickly settled areas in gigantic projects for conducting sewage to distant tidal waters or large streams. The perfection of land processes alone will not secure such results. I have expressed the hope that chemical processes will be thus perfected.

With Mr. Rafter's position I must squarely take issue. He apparently concludes that a land treatment would have been preferable to the dual system recommended and placed in operation in East Orange. With the general principle that land treatment is better than any other process of purification, where the conditions are favorable, I am in accord. But in his application to East Orange he has, beyond question, fallen into the error against which he cautions, viz.: "That no sweeping rules of

procedure governing every case arising can be laid down." My professional work has included tidal discharge, mechanical straining, broad irrigation, land filtration, chemical treatment and combinations of these principles, varying with the conditions treated. The combination process constructed in East Orange was not the result of blind infatuation with any method, but was recommended because a painstaking examination of the local conditions appeared to justify no other course. It is fair to assume that Mr. Rafter is not familiar with the local conditions surrounding the works, and has not fully comprehended some of the difficulties with which they have grappled, otherwise it would be hard to account for some of his arguments or deductions. To demonstate moonlight to be cheaper as an illuminant than gas or electricity, does not thereby render it universally practicable. Because the Lawrence experiments show that filtration through clean gravel give better results than other processes attempted, does not thereby demonstrate the universal applicability of that process.

It can, in my judgment, be clearly shown that land treatment alone would not have been entertained, and would hardly have been practicable in East Orange, but such is hardly within the scope of this discussion. I realize, however, that Mr. Rafter's position and conclusions are so far from my own that they require some further comment. He assumes that my knowledge of chemical treatment is sufficiently limited to lead me to conclude that the purity of the effluent will increase uniformity with the chemicals! His reason for this assumption appears to be my statement that 3 grains of lime and 2 grains of sulphate of alumina per gallon of sewage, as at present added, are too small to indicate the results obtainable by chemical precipitation. He then cites Mr. Learned's experiments, where 7 grains of sulphate of alumina per gallon removed 25 per cent. of the total solids in the sewage, and 14 grains removed 30 to 32 per cent. These experiments (using chemicals in each case in excess of those in East Orange) appear to support my judgment, that more chemicals should properly be used to relieve the labor of the land.

Mr. Rafter's estimates of land preparation as compared with the chemical process are in no sense applicable to the conditions met in East Orange. He fails to consider the cost of lowering Second River through the grounds (about $16 000) as part of the land preparation. He overlooks the fact that a very expensive building—needlessly expensive so far as the effectiveness of the process is concerned—is chargeable to the precipitation plant as now constructed. He neglects to allow for the excavation of the grounds before the sand and gravel are added, as he proposes; this excavation would be needed over 7 acres to a depth of 3 feet, since the out-fall sewer (grade 1 per 1 000) could not be raised and is now level with the surface. If this excavation were not made before the gravel was added pumping would be required. He proposes to use 19 acres while only 14 are available, the remaining 5 being pur-

chased with the original tract to serve to separate the grounds from numerous houses in Bloomfield.

My judgment is that the principles involved in the East Orange works are those most competent to grapple with the difficulties there existing; but I am satisfied that better results may be obtained from the precipitation without increased expense by changing the chemical process.

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TRANSACTIONS.

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AMERICAN IRRIGATION ENGINEERING.

By HERBERT M. WILSON, M. Am. Soc. C. E.

WITH DISCUSSION.

The development of the scientific practice of irrigation engineering in this country is a thing of to-day. Until about 1882, there can scarcely be said to have been constructed a single irrigation work designed on sound engineering principles. The art of American irrigation engineering has only been developed within the past few years, while the majority of the more modern and creditable works are but now approaching completion.

The early works of Colorado and California were all hastily constructed under great pressure from their owners, in order to realize some moneyed return at the earliest possible moment, and as a consequence the charges for maintenance and repairs have been so great that the financial results have been far below the expectations of their projectors. Experienced engineers were not employed on many of these works; county surveyors, or in fact any one who could run a level and

discover section lines, was employed to stake out the lines, while contractors or farmers designed the dams, regulators, and other works. The grades, velocities and capacities of the canals were matters of chance or guess, rather than computation. It was not until within the last decade, when such works as the High Line and Del Norte Canals of Colorado, the Arizona Canal of Arizona, and the Calloway Canal of California were constructed, that experienced engineers were employed to design such works. In fact it would be an error to say that the men who built those works were experienced irrigation engineers, for while most of them had much experience in other branches of civil engineering, irrigation practice was then an entirely new departure to them, and only now is the science of irrigation engineering becoming properly understood.

In the earlier works little attention was paid to the hydraulics of engineering, and the dimensions of the canals were more likely to be limited by their cost than by the volume of water available for diversion from the stream. In fact several large canals have been constructed where, had their capacities been but half that given them, they would have been large enough to discharge all the water perennially flowing in the streams from which they were taken. One notable instance of such an error is the Florence Canal in Montana, which has been constructed for 15 miles with a bed-width of 20 feet, a depth of 4 feet, and a capacity of 275 second-feet; while the south fork of the Sun River, from which it is diverted, rarely discharges during the irrigation season over 30 second-feet, to all of which there were prior appropriations by the older settlers; so that the bottom of the canal has never even been wetted since its construction in 1888.

Like everything which the Americans undertake, now that they have really begun to take an interest in irrigation development, they are bringing to bear upon it their proverbial push and energy; and the advance made in the number and magnitude of the works, is being kept pace with by the skill and intelligence displayed by engineers in overcoming difficulties, and in appreciating the science of irrigation engineering. The changes wrought in the practice of this science in the past few years are astounding; there are to-day under construction numerous irrigation works, both for the utilization of the perennial flow of the streams by direct diversion and for saving the storm flow of intermittent streams by means of storage. Of these a dozen canals are completed or under construction which have capacities varying from 1000 to 2000 second-feet,

with bed-widths of from 50 to 70 feet, the length of the main lines of which are from 50 to 100 miles, with as many more miles of laterals and distributaries. Such canals will irrigate from 100 000 to 150 000 acres each, and will render habitable twice that area, affording at an average of 40 acres to a farm, homes and support for 3 000 families each. Of storage reservoirs, there are half a dozen completed or under construction, which will impound from 800 to 300 000 acre-feet of water each, or sufficient to irrigate and reclaim nearly that many acres of land.

While not of such magnitude as those of India, the irrigation works of the United States are second in importance to those of no other country in the world, and in many respects compare very favorably with the Indian works. The area of land commanded by works either completed or under construction, is second only to that in India. Except the Cavour Canal in the Piedmont, in Italy, there is no work of this kind in Europe which compares in size with our more modern works.

There are four essential points of difference between the irrigation works of this country and those of India. The first relates to the ownership and legislation; the second, to engineering; the third, to construction; and the last to superintendence and maintenance. In India all land and all water is the property of the Government, and the irrigation works are designed, constructed, maintained and operated by the Government. In consequence of this the legal questions involved are comparatively simple; they relate chiefly to the amounts of water to be distributed to consumers, and right of way through improved land. The question of profit is not always paramount, and while the direct money return is often small, the indirect return to the Government is always large; in enhanced revenues from the rental of land, in immunity from famines and their consequent heavy drain on the Treasury for relief and charity, and in the general benefit to the Government resulting from increased resources and exports. In the United States, while the general Government benefits, as does that of India from the last cause, the owner of the irrigation works does not, as he is invariably a private individual or corporation. Then, as the lands are all private property, and the water the property of the public until appropriated, the constructors do not benefit by enhanced value of the land unless they purchase and own it. The priority of right to appropriate water and the ownership thereof, give rise to some of the most troublesome

and expensive legal complications with which the Western people have to deal. Direct money profit is essential to any irrigation project in this country, and in our most successful works, that has been chiefly realized from the sale and ownership of land, the value of which has been increased by furnishing it with a water supply.

As to the second point of difference, which relates to the engineering, the character of the Indian rivers and their relation to the irrigable lands is such, that the canals taken from them rarely require long or difficult diversion lines to bring the water to the lands. The chief engineering difficulties encountered are in the construction of stable diversion weirs and headworks in the sandy river-beds, and in contending against the enormous flood discharges of those rivers. In most portions of our country, good firm rock, or heavy gravel and clay soils can be found, in which to locate the headworks; while the relation of the rivers to the irrigable lands is generally such that long and difficult diversion lines have to be constructed before the water becomes available. In other words, the chief skill of the Indian engineer is in greatest demand in the location and construction of the headworks; that of the American engineer, in building the first dozen miles or so of his canal line. The only Indian canal of magnitude having a difficult diversion line is the Ganges Canal, a description of which was given by the author in an article entitled "Irrigation in India," published in Volume XXIII of the *Transactions* of this Society.

In the first 20 miles of the Ganges Canal four large flood-torrents are crossed, two of which are passed over the canal in masonry super-passages, one is admitted into it by a level inlet, while the fourth is crossed by an embankment or terreplaine and high aqueduct. Throughout its entire length, however, this canal is excavated in gently sloping or nearly level country. In the United States are numerons canals which are constructed for miles on steep hillsides and mountain slopes, often excavated in the sides of the rock cañons, requiring great lengths of rubble or masonry retaining walls to hold them up. Many deep and wide river valleys are crossed, some requiring aqueducts or flumes as great as 50 to 100 feet in height, while numerous tunnels have had to be pierced in the steeper slopes of the cañon walls. The most notable diversion line in the world is, perhaps, that of the Turlock Canal in California, which will be described at length further on. Only in extreme southern New Mexico, Arizona and California are the streams like those of India and

the dams, more difficult of construction, while the diversion lines are simpler.

The third point of difference, which is one of a structural nature, is chiefly due to the haste demanded and the cheapness required in the first cost of constructing these works. In India the works are constructed by the Government, and are accordingly designed with a view of making them so permanent that they will last nearly for all time, and that when once constructed the charges for maintenance and repairs shall be a minimum. To this end, all irrigation projects are carefully thought out and surveyed and resurveyed, until the best possible location has been chosen and the details of the scheme perfectly elaborated. The weirs, headworks, aqueducts and regulating gates are all constructed of the most substantial masonry, and the repairs to such works are reduced to a minimum. With us the main object is to reduce the first cost to the lowest possible figure, in order that the works may be completed quickly and cheaply, and the investors begin to derive some profit at the earliest possible date. The diversion weirs have generally been constructed of cheap wooden framing, and occasionally, as an unusual extravagance, of rock-filled crib-work; while many canals, especially in California, are taken directly from the streams by a simple inlet, with no dam or other diversion weir. Little time is spent in ascertaining the best location either for the headworks or the canal line, and wood is universally employed in the construction of flumes or aqueducts, regulating gates and falls. A canal may be run along the top of a fill rather than make a short detour to keep in cut, though the fill may be washed out by the first flood or be destroyed by the canal itself. A notable instance of the rapid construction of an irrigation canal is that of the Del Norte Canal in the San Luis Valley in Colorado. This canal is 60 feet wide at the bottom and 5½ feet deep. Over 30 miles of its main canal, and as many more miles of distributaries, in addition to the diversion weir and the various regulating gates and other works, were constructed in about one hundred and ten days in the winter of 1883. Many miles of this canal were surveyed and located with a construction party immediately following them. Fortunately the country is such that few mistakes were made, and the canal is a rare instance of such hasty work being done in a satisfactory manner.

The pressure which engineers are now bringing to bear upon the projectors of irrigation works is such, that more good, substantial and

permanent work is being done than at any time in the past history of irrigation development in this country, and the date is not far distant when our irrigation works will be designed and constructed with the care they deserve. There is even less excuse for faulty and unsafe work on an irrigation system than on a railway, for, should a railway bridge be badly constructed and give way, the lives of only those few persons who are upon the train which falls through it are lost, and if the line is badly located the chief loss to the company is in the deterioration of the rolling stock and added cost of haulage; but if a dam or similar work on an irrigation system gives way, not only are the lives of those whom the floods may engulf endangered, but thousands of dollars' worth of property are destroyed; and the lack of water for the irrigation of the crops may impoverish and render destitute hundreds of families, who are dependent upon the water supply to mature their crops. In addition, the bad location of the canal line and cheap construction of works, means such enormous outlays for repairs and maintenance as will consume all the returns from water rentals, and, perhaps, run the company into debt.

The fourth point of difference referred to between Indian and American irrigation works, is in the maintenance and supervision of these works. In India the fact of the completion of an irrigation system is no excuse for dispensing with the services of the engineers. A chief engineer, with his assistants, who have charge of the various divisions of a canal, are always retained in order to keep the works up, and to design and superintend improvements. In addition to the engineer corps, are overseers and patrols, the latter having comparatively short sections of the canal which they walk daily, in order to report the conditions of the works and make needed repairs, and especially to perform police duty by preventing damage being done by heedless or vicious persons, and to keep cattle from tramping over the ditches. The superintendence by these engineers and overseers renders it possible to keep the canal up to the highest state of efficiency, and the magisterial and police powers given the engineers and patrols, respectively, enable the canal officials to arrest and punish offenders against canal laws.

With us, after a canal system is completed, the services of the engineer are usually dispensed with, and the supervision and maintenance of the works falls to the lot of some superintendent, who may be a farmer, having no knowledge of engineering and as little of the needs of the work under his charge. Gophers are permitted to burrow at discretion

in the canal banks, and small leaks go unattended, as their dangers are unappreciated; while flumes and weirs and other perishable works are permitted to go unrepaired until they are past repairing and must be renewed. Rarely is any proper system of patrol in operation; sometimes ditch-riders are employed, though usually only on the more dangerous sections of the canal. On the vigilance and skill of the patrolmen and superintendent, largely depends the successful operation of a canal system, and this branch of the service is that which is most neglected and least thought of in the administration of American canals. One great canal in Kern County, Cal., that of the Kern Valley Water Company, is a startling example of the lack of proper supervision. This canal has a bottom width of 125 feet, a depth of 10 feet of water, and originally its grade was 1½ feet per mile, with falls in order to overcome the slope of the country. These falls were cheaply constructed and were soon undermined and carried away, and the canal was allowed to deepen its bed until now its grade is 4 feet per mile, and it is in places as much as 16 feet in depth below the surface of the country. Had proper supervision been kept up the falls could have been repaired before they were too badly damaged, and the exercise of engineering control would doubtless have induced the owners to replace them after they were carried away and before the canal-bed had been badly eroded.

In America, until recently, little attention was paid to hydrographic questions involved in irrigation construction. If it appeared from a cursory observation and a few inquiries that a stream discharged sufficient water for the needs of the canal company, a canal was constructed having a capacity to use more or less of that amount, and after it had been in operation a few years, it might be discovered that the average discharge of the stream was insufficient during years of minimum rainfall. In order that money may not be wasted in the construction of such works, it is essential that the discharge of the various streams should be known for a period of years. In general, no private company can afford to await the time necessary to ascertain with any degree of accuracy what this amount is. Within the past few years good work has been done to this end by public officers. The State Engineers of California and Colorado have gauged several of the more important streams in their States, and from these and the rainfall on their basins we are enabled to obtain rough data from which the run-off of their minor branches may be ascertained with some degree of accuracy. Within the past few years,

however, the Irrigation Branch of the United States Geological Survey has inaugurated some valuable and extensive researches in this direction, and had the Government appropriations permitted their continuance for a few years longer, these would have furnished us with results of inestimable value. As it is, these observations are still being conducted, though with less detail and thoroughness than would be the case were liberal appropriations available. Regular stream gaugings were conducted during the years 1889 and 1890, on most of the more important streams in the arid regions, and, as the former of these two years happened fortunately to be one of the driest in the history of our country, we are able to determine approximately what is the minimum run-off from the catchment areas of most of those streams; while comparison with observations conducted during a period of years on certain streams in Colorado and California, shows that the year 1890 was one of good average precipitation. In addition to this good work, observations were conducted to determine evaporation, seepage in different soils, and the amount of sediment carried in suspension during floods. From these it is now possible to approximately determine the loss from evaporation in storage reservoirs in the localities in which these observations were made. In this direction, however, there remains much yet to be done. Fortunately the directors of several agricultural experimental stations, which are operated by the United States Government in connection with the State Agricultural Colleges in the West, appreciate this, and experiments are now being conducted at Fort Collins, Colorado; Tucson, Arizona, and a few other stations, from which we may hope in the course of time to learn for those particular localities what are the annual and average amounts of evaporation, both from water surfaces and from soil; what is the duty of water on different soils and crops, and the percentage of loss in canals by absorption, in addition to other similar problems that await solution. To be sure, much work of this character has been conducted by eastern engineers in connection with the water works for cities; this is notably the case with the experiments on the Cochituate and Croton watersheds, but the results obtained are of comparatively little value when applied to the changed climatic conditions prevailing in the west.

The scope of this article is such that it is not possible to give in detail the results of experiments to ascertain duty, absorption, and the effects of alkalinity, as these differ greatly with different crops, soils

and climatic conditions. In an able article on "Irrigation," read before this Society in March, 1887, Mr. Edward Bates Dorsey gave much valuable information on these subjects, while the discussion provoked by the article produced as much more. It is well to note, however, that as the influence of capitalists and educated engineers is brought to bear on the cultivators, and as the experience of the latter is increased, the duty of water is constantly rising. This result is also largely due to the increased value of water and the saturation of the soil by successive years of irrigation. In Colorado, for example, State Engineer Maxwell and others, who a few years ago placed the duty at from 60 to 80 acres per second-foot, now claim that it is 100 acres, and a few believe it to be 125 acres per second-foot. In California, Wyoming and elsewhere, the same change of opinion is taking place; and while the ordinary duty of water in the San Joaquin Valley is estimated to be 110 acres, a duty of over 1 000 acres has been obtained in a few cases in Southern California by means of sub-irrigation from pipes.

A few years ago a great outcry was raised regarding the efflorescence of alkaline salts and the production of fevers due to irrigation. This, while true, is coming to be less feared as its causes are better understood. With an increased knowledge of the necessity of natural or artificial drainage as an adjunct to any well-planned irrigation project, and a better understanding of the proper mode of locating and constructing canals, so as to produce a minimum rise of the sub-surface water level, the damage caused by these evils is reduced to a minimum.

By far the most important and interesting problem in connection with irrigation which yet awaits solution, not only in our own country but wherever in the world irrigation is practiced, is that of securing a uniform and correct standard or unit of measurement of water. At present water is not sold, as it should be, like other commodities which have an intrinsic value per pound, yard or gallon, though it is appreciated by every engineer and canal owner throughout the world that such would be the most satisfactory method of disposing of it; but owing to the difficulties of measuring flowing water at a cost commensurate with its value, some other device has always been employed, such as charging a higher rental, or a higher cash selling price, for lands to be irrigated; or putting a charge of so many dollars per acre on the land served, as a water-right, or by some similar and equally unsatisfactory method, whereby the amount of water going with the land is not specified and

remains always a point of contention between the purchaser and vendor. On this subject Mr. Walter H. Graves, C. E., one of the prominent irrigation engineers of Colorado, says:

"The difficulties attending the measurement and allotment of water by reason of the constantly varying conditions and fluctuating head, has given rise to a great variety of methods and customs; and in this respect where system is all important and most needed, the absence of it is conspicuous, and all efforts for the establishment of a correct and universal system have thus far been defeated. The most common practice in Colorado is by a sort of nondescript 'inch' unit, taken from a method of measuring common among miners in placer diggings called 'the miners' inch,' this in turn is taken from a module or unit of measure called the *oncia magistrale*, in vogue in the irrigating provinces of Italy, that was devised some three or four centuries ago by one Soldati. It is rather a method of subdivision than one of measurement. It is called the 'statute inch' from an attempt to prescribe it by State statute; the very provisions of which, however, make the incongruity manifest. In practice it is impossible for the consumer to know how much water he is using. There is a great need throughout the entire irrigating region of a simple, practical, trustworthy apparatus for the measurement of water. The water meter employed in the water-works of cities is too complicated to apply to the open ditches, where it may be choked with mud and weeds. The ingenuity of some engineer will be amply repaid for the invention of a hydrometric sluice that will meet the requirements of this demand."

A new water meter for irrigation, devised by Mr. A. D. Foote, and described by him in a paper read before this Society in March, 1887, is perhaps the best form yet suggested, and a modification of this employed by Mr. Graves on the canals of the San Luis Valley, in Colorado, seems to work with some degree of satisfaction. Next to this the common V measuring weir is one of the best devices employed.

Perennial Canals.

Of the older type of canals constructed in our country it is necessary to say little. Both in Colorado and California, irrigation by Americans as distinguished from that practiced by the original Mexican inhabitants for several centuries past, came into vogue in the latter part of 1860; when some of the early pioneers abandoned gold digging for the more profitable industry of agriculture, and when in California, especially, they discovered after repeated failure that the rainfall was deficient for crop raising. The practice thus forced upon them was at first compara-

tively easy of accomplishment, for in many localities the mining ditches which were ceasing to return profit from disuse, were ready at hand for the transportation of water to small garden plots and fields in the foothills of the gold-mining region, and at once brought home a blessing in disguise. In many parts of California and Colorado the early pioneers in irrigation are known now on account of their great wealth, most of which was realized through the aid of irrigation. For several years the practice of irrigation was on a moderate scale. At first, the water was diverted from the smaller streams, which rendered it easily accessible by the individual efforts of the poorer farmers, and each irrigator owned his own separate ditch, was his own engineer, and himself constructed the works. The dams and head-works usually consisted of brush or stone thrown across some narrow channel; and the canal, of a straight cut, without any regulating gate to control the water, badly aligned, having accidental slopes and no pretence at uniformity of cross-section or grade. As the water was easily obtainable, however, such works proved profitable and beneficial. As the water of these smaller streams became all appropriated, and later settlers were compelled to look to larger streams for their water supply, co-operation for the construction of larger weirs by regular ditch companies became a necessity. Of these earlier works of greater magnitude, among the first constructed were the Fresno, King's River and San Joaquin Canal, and a few others built in the San Joaquin Valley.

The first of these was constructed in 1872 by Mr. Isaac Friedlander of Fresno County, under the direction of Mr. Alfred Poett, Civil Engineer, and was perhaps the pioneer of this kind of work in our country. The Fresno Canal is diverted from the Fresno River by means of a timber dam, 311 feet long, which raises the water 6 feet above its original level. It was constructed of two rows of main piles, from 20 to 35 feet long, planted 10 feet apart and driven firmly into a stratum of clay. Between the piles was a double row of 4-inch sheet piling, and the space between the two rows of piles was filled in solid and planked on top. Below the toe of the dam was a timber apron of 4-inch plank set at an incline, from which the water passed on to a thick layer of loose rock. The head-works of the canal consisted of a sort of flume or box of timber 30 feet wide and closed by six gates. The canal had a uniform bed-width of 20 feet with side slopes of 1 on 2, the depth of the canal being 8 feet and the depth of water 6 feet. Where they were

in embankments the top width of the canal banks was 4½ feet. The grade of the canal as originally designed was 6 inches to the mile, and it was designed to give a velocity of 2 feet per second and a discharge capacity of 360 second-feet.

The heads of the distributaries consisted of an arrangement of fluming or boxing closed by simple sliding gates. The main distributing ditches were 10 feet wide at the bottom, slopes 1 on 2, top width of banks 2 feet, and height 2 feet, with varying grades, the maximum grade being 33 inches to the mile and the velocity about 2½ feet per second. In designing the above work it was estimated that this canal would irrigate 72 000 acres, and the duty of the water was expected to be from 200 to 400 acres per second-foot. It is an interesting and notable fact that, whereas the duty of these earlier canals in California did not rise much above 80 to 100 acres, experience in its use and its increased value are daily causing this duty to increase, so that now it not only reaches the earlier estimates but in many cases even passes them. Among the weak points in earlier irrigation construction were the steep grades and high velocities given, and practice is now bringing these down to the theoretical dimensions designed for the Fresno Canal. The earlier canal works of California were faulty in many respects, but chiefly in the location and construction of their head-works. In these, if any weir at all were built, this was done in an unsubstantial and cheap manner, causing great loss both in the operation of the canal and in the necessarily frequent reconstruction of the weir. In all cases the alignment of these canals was faulty; sometimes sharp bends were made, which greatly retarded the flow of the water and caused the deposit of sediment and erosion of the canal banks, while other constructors went to the opposite extreme, giving great broad curves, such as would be necessary on a railway line. No provision for the discharge of flood drainage was made, and the canals were frequently destroyed, owing to their gathering local flood waters for which no relief was provided; while grades were equally faulty, the chief error being in making them too steep.

The next stage in the development of canal evolution is represented by such works as the Del Norte and Highline Canals in Colorado, the Calloway Canal, in Kern County, Cal., and the Arizona Canal, in Arizona. Though these works were in general fairly well aligned, and while intelligent thought was brought to bear on their design, they show, however,

PLATE XXVI.
TRANS. AM. SOC. C. E.
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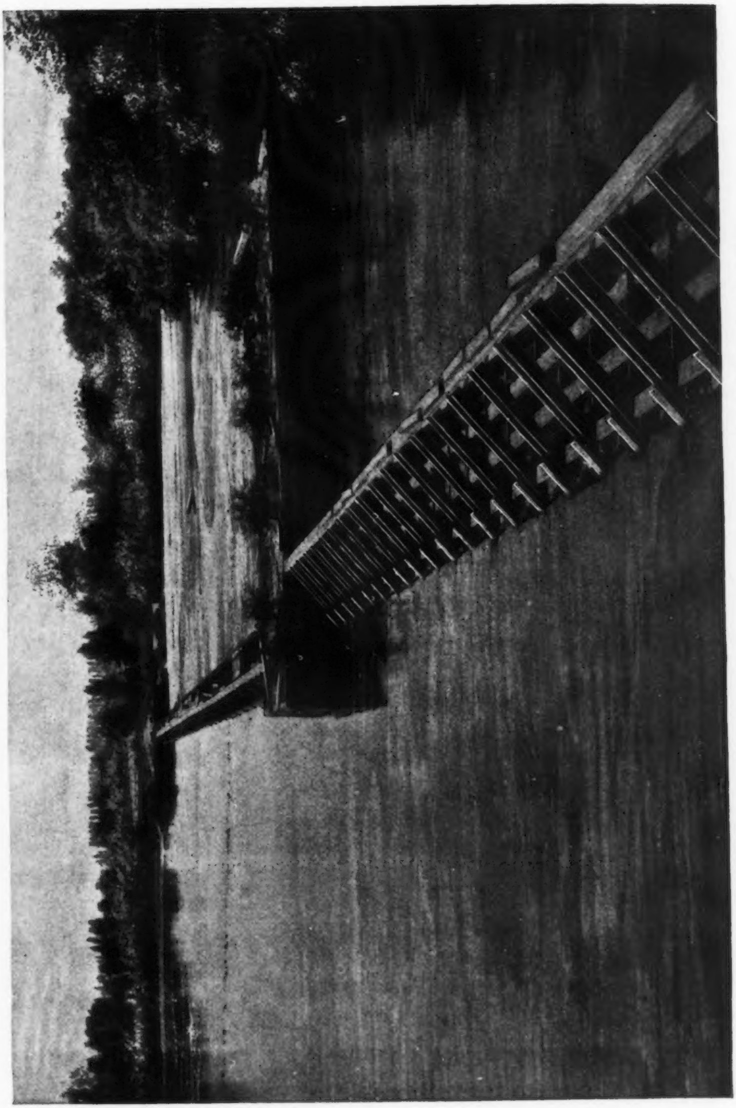
WILSON ON AMERICAN IRRIGATION.



KERN RIVER DIVERSION WEIR, HEAD OF CALLOWAY CANAL, CAL.

a lack of experience on the part of the engineers constructing them and of the capitalists furnishing the money, which was due to the newness of this class of work. Their chief fault lies in the perishable character of the work done upon them, especially in their head and regulating works, which are of the most temporary character.

The Calloway Canal is diverted from the north bank of the Kern River, a short distance above the town of Bakersfield. It was located in 1875, and appropriates 1 476 second-feet of water, though its capacity is little over half of this. The Kern River is the third largest in the San Joaquin Valley and rises in the high Sierras. Its headwaters drain the base of Mount Whitney, whose melting snows insure it a perpetual discharge of some magnitude. The average discharge of this river is about 2 700 second-feet, while the average maximum discharge during the rainy season is 19 000 second-feet. The water of the canal is diverted from the river by means of an open wooden weir 400 feet in length and extending across the entire width of the river from bank to bank at right-angles to its course. This weir (see Plate XXVI) rests on three rows of 4 x 12-inch anchor piles, driven 10 feet into the river bed, and two rows of 4 x 12-inch sheet piling parallel to the course of the stream. There are one hundred rows of these piles across the bed of the stream and upon them rests the superstructure, consisting of as many bays, 4 feet in width between centers, and composed of beams or rafters set at an angle of about 40 degrees facing up-stream, and 50 degrees down-stream. On the bed of the river between these bays and resting on mud-sills laid on the tops of the piles, is a 2-inch flooring about 30 feet in length laid with the direction of the river. On the upper sloping face of these open timber bays are set grooves into which slide 2-inch planks, or as they are commonly termed "flashboards," and these are inserted in sufficient numbers until at low water they fill the weir from base to crest, forming a continuous closed dam. At high water and in flood times these planks are removed, one at a time, thus increasing the water way of the flood to any desired amount. The total height of the weir is 10 feet above its floor, and adjacent to its north end and closing the head of the canal, which is diverted at this point, is an open wooden regulator constructed in almost exactly the same manner as the weir, but exceeding its height by 1 foot.

For the majority of its length the Calloway Canal is excavated in a light, sandy loam, which is exceedingly rich and fertile, but of so light a

character for construction that the slopes given the banks are necessarily low. The canal line skirts around the slopes of the foot-hills to the northward on a changing grade, the object of which is to diminish its discharge as the various distributaries are diverted from it, while leaving it the same cross-section in order that it may ultimately be enlarged. The change of grade is from 0.8 feet per mile to the tenth mile, to 0.6 feet, then 0.4, and then it is level near the end. The capacity of the canal at its head is 700 second-feet, its bed-width is 80 feet, depth 5 feet, depth of water 3½ feet, and maximum velocity 2½ feet per second. Throughout its length the canal is built half in cut and half in fill. Its side slopes are 1 on 3 inside and 1 on 2 outside, though owing to the light, sandy character of the soil, these have changed until now the interior cross-section is an easy curved slope from the top of the bank to the center sub-grade. The canal is 32 miles long, and at its second mile a double escape and regulating head is constructed for the purpose of discharging all the waters at this point in case of necessity. This consists of weirs which, like all the others on its line and that of its distributaries, are constructed of wood after the general design described for the head weir and regulator. In the thirtieth mile, Poso Creek, the first principal drainage encountered, is admitted to the canal by a level inlet, while in the opposite bank of the canal a wooden outlet weir is constructed of the design above described. Diverted from the Calloway Canal are some sixty-five distributing ditches, from 8 to 20 feet bottom width and from 1 to 9 miles in length, their aggregate length being 150 miles. These branches will carry 3 feet in depth of water, and have slopes the same as that of the main canal and a grade generally of 1.6 feet per mile. This canal commands 200 000 acres of land and most of the water is now employed for the irrigation of the numerous ranches below it. On the line of its distributaries, which run down the slope of the country, numerous falls are constructed, their design being similar to that of the head weir, and their object being to maintain a uniform grade throughout the canal and keep its waters as near the surface of the irrigable land as convenient.

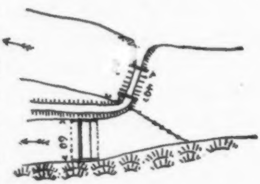
There are some peculiar features connected with the construction of the Del Norte Canal, which is situated in the San Luis valley in Southern Colorado. This canal takes its supply from the Rio Grande at a point just north and west of the town of Del Norte, and after skirting the

foot-hills north of the river for a few miles, it is located around the northwestern edge of the valley, and commands most of the lands between the Rio Grande and Saguache Creek, of which it will irrigate about 225 000 acres, though the area commanded is much larger than this. The discharge of the Rio Grande, as shown by gaugings maintained by the hydrographers of the United States Geological Survey at Del Norte, averages about 1 250 second-feet; and while its minimum discharge has been as low as 400 second-feet in the month of August, its maximum has reached at least 5 000 second-feet.

One of the points of interest in the construction of this canal was, as before stated, the rapidity with which this work was done. In the four winter months, from December to April, 1883–84, something over 1 400 000 cubic yards of material were excavated to form the channel, requiring the employment of nearly 5 000 laborers and 1 200 teams, all in a sparsely inhabited country. The location of the headworks was not the best; they should have been placed at least 10 miles further down the river, according to the statement of Mr. Walter H. Graves, the engineer who built them. As the canal was projected by the citizens of Del Norte, local pride prevailed over good sense, and decided upon their situation adjacent to the town. The excessive grade consequently given in the first few miles of this canal is another curious and noticeable feature of its construction. To have adopted a grade dictated by good judgment would have rendered its construction impracticable, as it would have climbed to the tops of the adjoining mountains before reaching the lands to be irrigated, so great is the fall of the country to the north and east, hence the line skirts the base of the foot-hills for the first 12 miles, and the canal is given a fall corresponding with the natural slope. This grade is excessive for so large a canal, averaging about 8 feet per mile until it emerges from the foot-hills, where it is reduced to about 1 in 2 112. It was anticipated that though the material through which this canal was constructed was entirely of coarse gravel, drift and rock, it might be necessary in time to revet its channel with rock, as no falls were put in it. Fortunately this has proven to be unnecessary. In one place the grade is 35 feet per mile for a quarter of a mile, but as the canal here is straight little damage has been done by this high velocity.

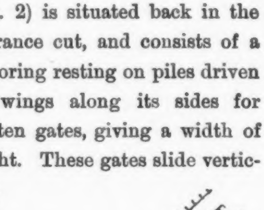
The diverting weir, of which a sketch plan is shown, is located at a point where the Rio Grande flows through a comparatively narrow

gorge having firm earth banks on one side and a rock bluff on the other. This weir has a total height above the grade of the river bed of 5 feet 6 inches. It consists of a series of open bays which are closed by flash-boards, and is almost identical in construction with the weir at the head of the Calloway Canal, though a little more substantial than that weir. Its total length is 80 feet, and it contains seven openings of 6 feet wide each, the remainder of the weir consisting of a rock and earth embankment, over which it is not intended that the flood waters shall pass. As there



are several separate channels to the Del Norte River just above the canal-head, it has been necessary to construct wing dams and other obstructions for the purpose of training the main body of the river into the single channel which passes by the head of the canal. This is done by means of wing dams of earth or rock, and in one case by the construction of a large crib dam entirely closing one channel. This dam is constructed of 12 x 12-inch crib work, drift-bolted and wired together and filled with rock. Its height varies from 6 to 15 feet, while its length is 1 100 feet.

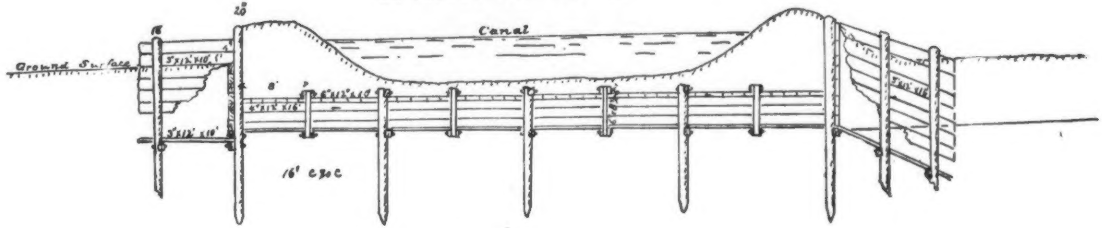
The regulator (see Plate XXVII, Fig. 2) is situated back in the canal a hundred feet or so below the entrance cut, and consists of a wooden structure having a heavy plank flooring resting on piles driven in the canal bed, and having wooden wings along its sides for protection from erosion. It consists of ten gates, giving a width of 59½ feet of clear opening, and 8 feet in height. These gates slide vertically between guide posts, and are operated from above by means of screws turned with hand levers. For the first section of the canal below the head the bed-width is 60 feet, depth of water 5½ feet to grade, below which is a sub-grade of 1 foot. Its maximum capacity is 2 100 second-feet, which is the second greatest of any canal



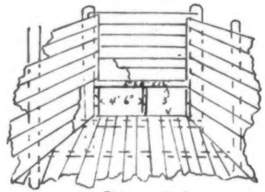
in this country. Eight thousand feet below the head is a bifurcation dividing the canal into two equal branches, each having a bed width of 40 feet. This bifurcation, of which a diagram is shown, consists of wooden planking protecting the outer sides and the dividing point

BOX CULVERT; DEL NORTE CANAL

LONGITUDINAL SECTION



END VIEW



Upper End

CROSS SECTION AND ELEVATION

REGULATING GATES DEL NORTE CANAL

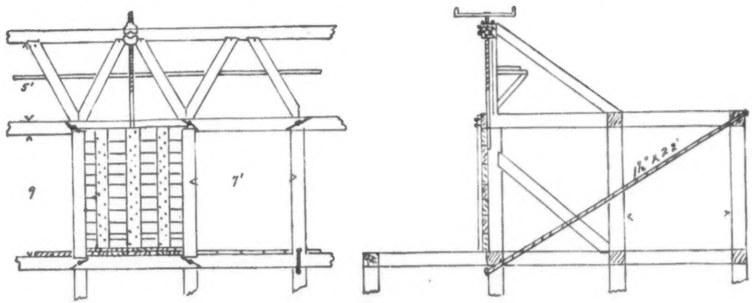


PLATE XXVII.
TRANS. AM. SOC. C. E.
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WILSON ON AMERICAN IRRIGATION.

between the two canals, and of a wooden flooring resting on piles. In the head of each branch is a regulator by which the discharge into either can be controlled, and each regulator consists of five gates of the same general dimensions and form of construction as that at the head of the main canal. Lower down in the south branch is another bifurcation similar in character to that above, while numerous distributaries and smaller laterals are taken from both branches in such manner as to command most of the land below the canal and supply the private ditches. The fall in the majority of the branches of this canal is great, being from 10 to 15 feet per mile. When in gravel this has caused little damage, but the majority of these distributaries are in earth excavation or embankment, and have sustained much damage by erosion, necessitating the rip-rapping of large portions of their length. The engineer in charge of the work says, however, that it is cheaper to do this than to give the increased cross-section necessary with a less grade and velocity.

On the line of this canal are several works interesting in their design, but simple in construction. One is a wooden aqueduct whereby the Farmer's Union Canal, a separate irrigation channel, is carried over the south lateral of the Del Norte. The grade of this canal is kept high enough to give a clear crossing of the Del Norte Canal, immediately after which the water is dropped to the general level of the surrounding country, in two falls of 3 feet each, constructed of wood. In another place a smaller canal is taken under a branch of the Del Norte by means of a wooden box culvert or inverted siphon. (See Plate XXVII, Fig. 1). This culvert is founded on piles, and contains two openings of 3 x 4½ feet each lined with 4-inch planking, and covered on top by an additional depth of 6-inch timbers to support the weight of the superincumbent earth and water. The cross-section of this canal is unlike that usually employed on canals in other countries, and in most parts of this country, owing largely to the light and sandy soil in which it is excavated. The banks have no set top width nor fixed outer slope, though this latter is about 1 on 3; the inner surface has side slopes of from 1 on 2½ to 3, and is made deepest in the center, while the bed is not level, but is sunk to a sub-grade of about 1 foot in depth.

The Highline Canal of Colorado is another of the earlier canal structures of this country. It is diverted from the South Fork of the Platte

River, about 21 miles above Denver. Whereas the maximum flood discharge of this river is about 2 000 second-feet, its minimum flow averages as low as 110 second-feet; and, as a consequence, though the cross-section and discharge capacity of this canal is large, the water supply is not equal to the demand during a majority of the irrigation season. This fault could, however, be easily remedied by the construction of storage reservoirs on the Upper Forks of the Platte River. The canal commands an area of about 90 000 acres of excellent irrigable land in the neighborhood of a populous city, and should be one of the best canal properties in the country if its water supply were but secured against drought. This canal was designed and constructed by Mr. Edward S. Nettleton, though Mr. George G. Anderson, who has been the engineer in charge of it for the past several years, has made some marked changes and improvements in it, especially in reconstructing more permanent head-works.

The diversion weir is 14 feet in maximum height, 117 feet long and 6 feet wide on top. It consists of crib-work filled with rock, and whereas the up-stream slope is very flat, down-stream the fall is nearly vertical. Near the left bank is constructed a substantial masonry undersluice, open the entire depth of the weir, whereby the water in the river above may be drawn down to any desired level. The sluiceway consists of four iron gates, each 4 feet wide between centers and 7 feet in height. These gates are raised by means of screws, and the sills of the gates are placed at a distance of 2 feet below the level of the head-gates of the canal, in order to permit of sluicing out sand, etc. The canal is diverted from the right bank just above the weir, the head-regulator consisting of a set of five gates, each 6 feet in height and 3½ feet wide between centers. This head-regulator is constructed between solid rock walls and on a rock bed, and the gates, which are of wood, are raised by screws from a platform above. Immediately below the regulating gates is a tunnel excavated in granite, 600 feet long, 20 feet wide and 10 feet in height in the center; the lower end discharging into a great wooden bench-flume, which skirts the steep rocky slope of the Platte Cañon for a distance of 2 640 feet, where it terminates in the open canal excavation in which the water is carried throughout the remainder of its course. At the head of this flume (see Plate XXVIII) and at the lower end of the tunnel is an escape by which the water carried in the canal can be regulated. This escape consists of five gates, each 3 x 4 feet, of wood, and raised

BENCH FLUME ON HIGHLINE CANAL, COLORADO.

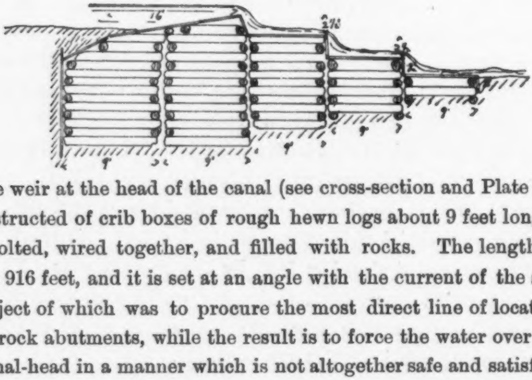
by rack and pinion, while immediately below these gates, across the entire width of the flume, are a set of check or flash-boards about 2 feet in height, which act as a sand-gate, by causing the deposit of silt immediately above them, whence it can be scoured out through the escape gates.

The great bench flume which extends for over half a mile below the tunnel, is 28 feet wide and 7 feet deep; its grade is 5.28 feet per mile, and its discharge, which is the same as that of the canal proper, is 1 184 second-feet. The main canal is about 85 miles long, and bifurcates at the 51st mile, one branch being 14 miles long, and the other 20 miles long. At its head the main canal is 40 feet wide on the bottom, and from 7 to 8½ feet in depth, with slopes of 1 to 1 in cut, and 1 on 2 in embankment, and with a uniform grade of 1.76 feet per mile. The line of this canal is crossed by several side gulches or torrents. In the first mile is an embankment 18 feet in height at a gulch crossing, but as the discharge of this is small, there is no inlet nor outlet dam constructed for the regulation of its flow, the water being admitted into the canal. In the third mile, Willow Creek is crossed by the construction on the lower side of the canal of a heavy earth embankment, which is 30 feet high and 8 feet wide on top, with slopes of 3 to 1 inside, and 2 to 1 outside. Above and below this embankment, escapes are constructed in the canal banks, which discharge directly over the natural gravel slopes of the country into Willow Creek. These escapes have lengths of 200 and 300 feet respectively. At the 9th mile, Plum Creek is crossed by a flume 918 feet long and 18 feet in height on a high trestle, founded on eight rows of 12-foot piles. In this flume are four sets of escape gates spilling into Plum Creek, and by which the discharge of the canal can be regulated. Originally a less substantial weir existed at the head of this canal, which was washed out and replaced in 1884 by the present weir. The total cost of both weirs, flume, ditch and other works was about $650 000, and the maintenance, cost, charges, etc., amount to about $25 000 per annum, including salaries, office expenses, etc.

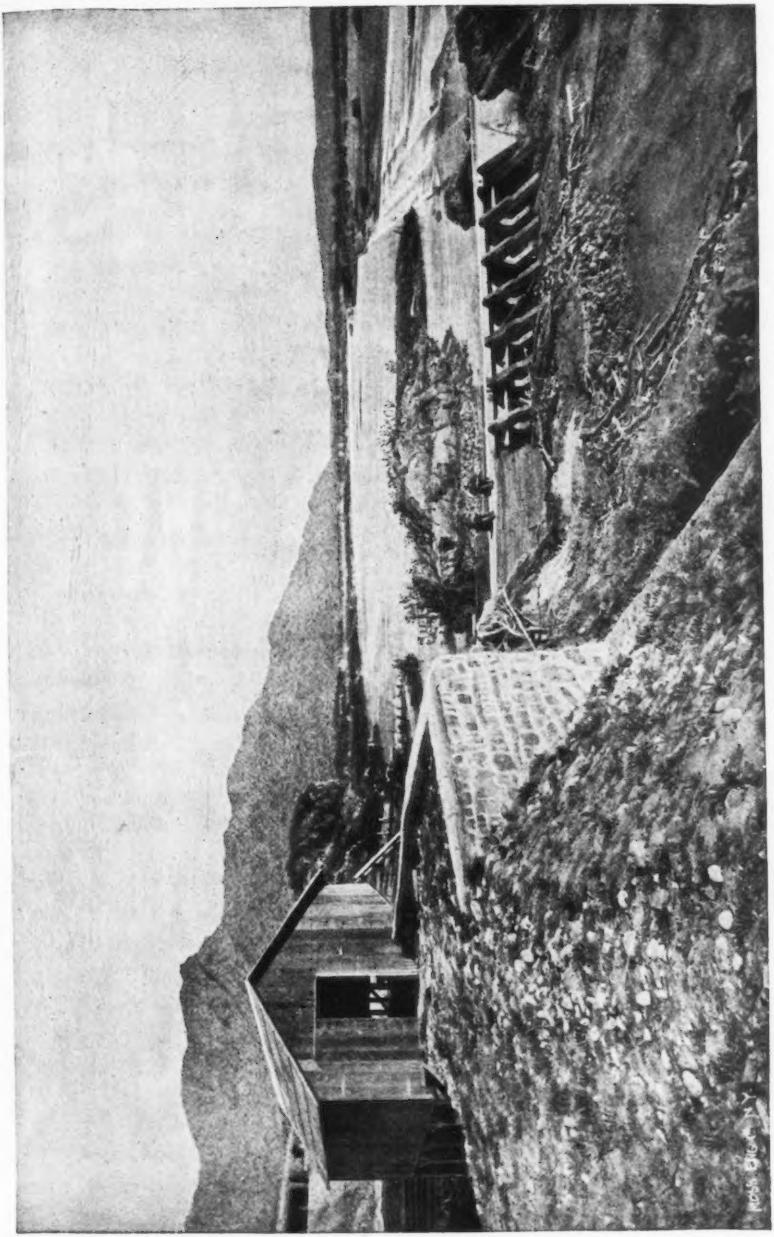
Another equally fine example of the third stage of canal construction in this country, in which stage are included all those large canals which have been put in operation during the past few years, and similar in most of its features to the works just described, is the Arizona Canal, which is diverted from the Salt River in Southern Arizona, about 25

miles above the town of Phœnix. The head of this canal is situated at a point 1 mile below the junction of the Salt and Verde Rivers, where the river leaves the Lower Cañon, and flows between a reef of rocks which affords an excellent site for the weir and headworks. Thence its course is along the north bank of the river, skirting the foothills surrounding Salt River Valley, and commanding an immense area of the finest quality of rich arable land; where, owing to the semi-tropic character of the climate, abundant yields of hay, grain, vegetables and such fruits as plums, peaches, grapes and oranges are cultivated. The Salt River is an extremely difficult stream from which to divert a canal, owing to the irregularity of its discharge, for while its average minimum flow is usually as low as 500 second-feet in midsummer, the greatest recorded flood was 300 000 second-feet, and each and every year floods from 10 000 to 20 000 second-feet occur. As a consequence of this erratic discharge, the river bed itself is very wide, and a long and expensive diversion weir is required in order to procure stability and permanency.

CROSS SECTION
ARIZONA WEIR



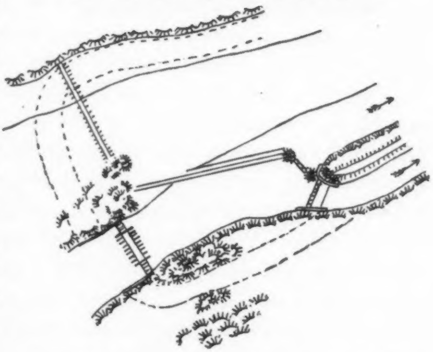
The weir at the head of the canal (see cross-section and Plate XXIX) is constructed of crib boxes of rough hewn logs about 9 feet long each, drift-bolted, wired together, and filled with rocks. The length of the weir is 916 feet, and it is set at an angle with the current of the stream, the object of which was to procure the most direct line of location between rock abutments, while the result is to force the water over toward the canal-head in a manner which is not altogether safe and satisfactory. The total height of the weir from the bed of the river to its crest is 10 feet for a little over half its length from the end adjacent to the canal-head, and 11 feet for the remainder of the way. The total length of the weir and apron parallel to the course of the stream is 48 feet, and in the deepest part of the gravel bed of the river the depth of crib-work is 33 feet. The up-stream side of the dam is planked, as is its entire facing,



WEIR ACROSS SALT RIVER, AND HEAD-WORK OF ARIZONA CANAL.

and is on a flat slope inclining backward from the crest; while sheet piling is driven in above the crib-work on the upper face. Down-stream the crest of each successive row of cribbing, as shown in the accompanying diagram, is from 2 to 2½ feet lower than that above it, causing the weir to fall away in broad steps, each of which is about 9 feet in length horizontally, thus breaking the force of the falling water and diminishing its tendency to destroy the weir; the last and lower line of crib-work being practically a swinging apron resting on the river bed and wired to the crib-work above it. This weir has proven very substantial, and has withstood for four years the enormous flood volumes above indicated. In the spring of 1891, however, the big flood of 300 000 second feet did much damage to it, and a new weir is now being constructed at the upper site, described further on. Beyond the east end of the weir (that furthest from the head of the canal), is an old channel of the river, separated from the present channel by a narrow ridge of rock and gravel, a few yards in width. This old channel is blocked by an embankment of gravel 10 feet in height, rising to an elevation a little above that of the crest of the main weir. The location of the head-works is not the best that could have been discovered, but their choice was forced upon the engineer, Mr. Sam A. Davison, owing to the limited time at his disposal in which to construct them and the smallness of the funds. By the selection of the present site he was enabled to build the works in much less time and at less expense than by the choice of a better site. It is proposed, however, to at once re-locate the dam at a point a couple of hundred yards higher up the stream, where a shorter weir can be constructed between better abutments and on a rock foundation. This will enable the weir to cross the river at right angles and permit the head of the canal to be diverted at such an angle from

PLAN
ARIZONA HEADWORKS



the weir as will give a clear scour past the regulating gates, thus reducing the pressure on the head and avoiding the objectionable deposits of silt in front of the gates which now occur. (See diagram.)

At present the head of the canal is diverted at a point about 50 yards above the end of the weir from the right or western bank of the river, and is separated from the end of the weir by two rocks which project above the water surface, and between which a waste or scouring gate has been constructed, the object of which is to prevent the deposit of silt in front of the canal-head. The relative position of these works is such, however, that quite an island has formed almost in front of the head-gates, which interferes with the free flow of water into the canal. The scouring sluice between the weir and the regulator is a simple structure constructed of wood, founded on piling and closed by flash-boards, and is very similar in its general design to those described for the open portion of the weir at the head of the Del Norte Canal. The regulating gates at the head of the Arizona Canal are well and substantially constructed of wood, and over the top of them crossing the canal from bank to bank, is a covered bridge giving ample room in which to operate the gates. These latter are eight in number, and are each 4½ feet wide by 6 feet high, though the total height of the regulator head is about 11 feet in order to prevent its being topped by extreme floods which bear with considerable pressure against the gates. Of the regulators six are raised by simple hand-levers, which are inserted into holes in the upright rods attached to the gates, and are used when the river and the pressure are low. Two of the gates, however, are operated by screws ingeniously geared by cog-wheels in such manner as to be operated by a winch turned by hand, and can be worked by one man under the greatest head of pressure.

At its head, and for the first considerable portion of its course, the canal has a bed width of 36 feet, carries a depth of 7½ feet of water and has a capacity of 1 000 second-feet. It is laid out with a uniform grade of 2 feet to the mile, its banks having slopes of 1 on 1½ in rock, and its cross-section being similar to that of the Del Norte Canal, with no berme, and a sub-grade of about 1 foot in depth. The main line of canal is 41 miles in length and it is now being extended. For the first 3½ miles below its head, excavation is entirely in rock or gravel, in places the gravel cut being 25 feet in depth. Below this heavy work the canal is entirely in earth on very gently sloping side hill, and is built half in excavation

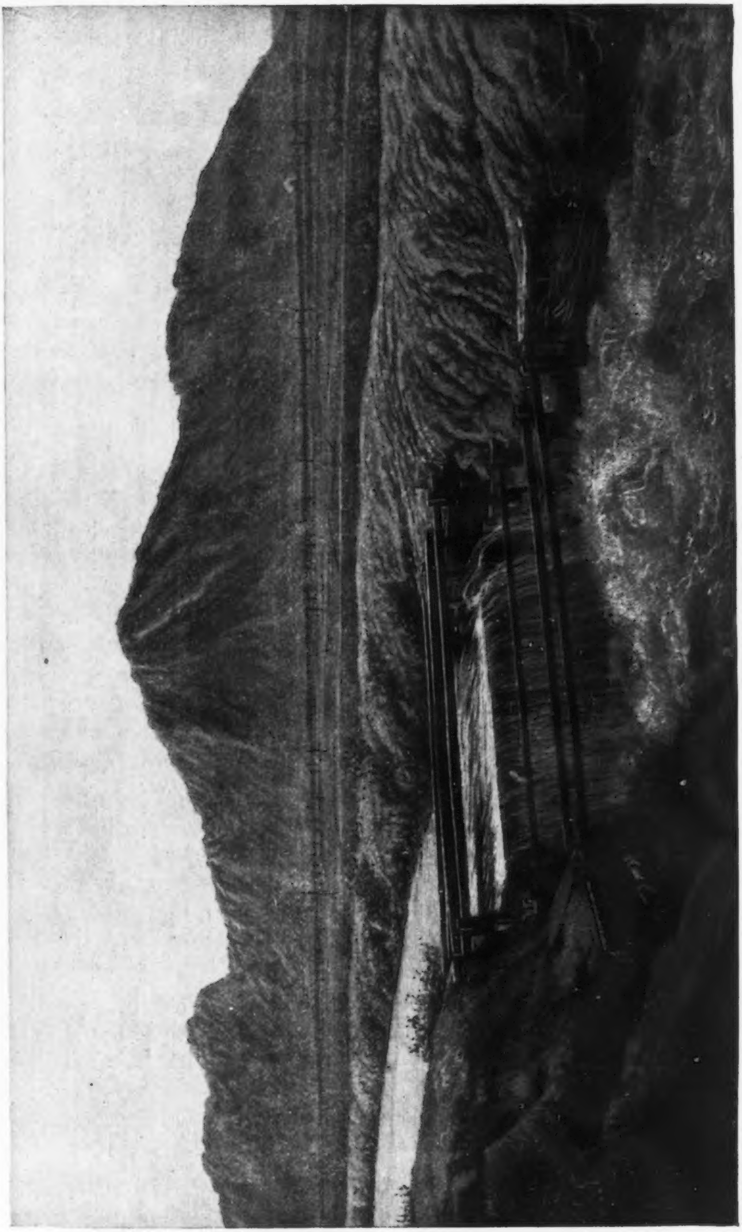
and half in embankment, with the exception of one short rock cut just above the diversion of the cross-cut canal. This cut is 15 feet in depth and has a fall built in solid rock 15 feet high, constructed both in order to drop the grade and to avoid some rock excavation, and also with a view to being used for furnishing water-power to be employed in the town of Phœnix. About 1 mile below the head of the canal an escape has been constructed, by which the discharge water can be let back into the river through a waste channel having a bed width of 40 feet and a length of 800 feet. The escape regulators consist of a row of gates having a total width of 40 feet, a length of 80 feet, and a depth of 12 feet, with a strong apron at the upper end extending at an angle of 45 degrees downward for 12 feet into the bed of the canal. Across the canal itself, below and adjacent to the escape gates, is a set of regulating gates constructed in a similar manner to the escape gates; the whole work rests upon piles, is well floored, and has stout wings to protect the banks. Well planned provision is made for sluicing out and getting rid of any sand which may deposit in the channel of the canal, and of emptying the canal in case of accident below.

After it reaches the earth excavation below the heavy work, the canal has a uniform grade of 1 foot to the mile and is 30 feet wide on the bottom for a distance of 26 miles, having an 8-foot berme on the embankment side, the banks having slopes of 1 to 1 in cut, and 1½ to 1 in embankment. In all cases the top width of the bank is 8 feet, and its crest is from 6 to 8 feet above the bed of the channel. All fills have extra widths and heights, giving them a secure appearance, while the curves are generally made very favorable and have been laid out with some attempt at intelligent alignment. In the first few miles the canal crosses several small drainage streams which have flood discharges of from 30 to 200 second-feet. These streams are admitted by simple level inlets, the water being passed on down the canal without any provision for wastage or escape, which is an undoubted defect in the designing of the work. Beyond Phœnix is a broad, level stream bottom of considerable extent, through which Cave Creek finds its way, having a flood discharge of at least 1 000 second-feet. At present owing to the difficulty and expense of confining this stream to a single channel, no provision is made for passing its flood waters, and when floods occur they inundate a considerable area of country surrounding the canal on both sides, and necessitate upon their subsidence the reconstruction of quite a portion of the canal banks.

In the 12th mile from the head is a flume 1 200 feet long spanning Kramer Creek. This flume rests on oak pile bents capped with heavy timber, and is well constructed and braced.

In addition to the 41 miles of main canal now existing, there are two main feeder branches which were constructed independently of the Arizona Canal, parallel to its course, and lying at distances of a few miles below it down the slope of country between it and the Salt River. The Arizona Canal Company have, however, purchased these, and use them as a portion of their system, the main Arizona Canal acting as a high line feeder to these auxiliary canals. At about the 20th mile of its course the cross-cut feeder is diverted from the main canal by means of two substantial sets of regulating gates, one of six openings in the main canal and the other of five openings at the head of the cross-cut. These regulators are founded on sheet piling and are constructed of wood, the gates being operated from an overhead bridge by means of hand-levers. The total length of the cross-cut is about 4 miles, the bottom width 22 feet, slopes 1½ to 1, grade 2.65 feet per mile, and capacity 375 second-feet. As it is built straight across the country down its slope, the total fall in its 4 miles is 128 feet. This drop is compensated by twenty-eight falls averaging about 5 feet in height each, and substantially constructed of wood founded on sheet piling. The checks in these falls (see Plate XXX) consist of flash-boards let in between posts, below which is an apron of wood 12 to 16 feet in length, while the banks of the canal are well protected by wings and sheet piling. The first parallel feeder canal supplied by this cross-cut is the Grand Canal, below which the dimensions of the cross-cut are slightly diminished until its terminus is reached at the second feeder, the Consolidated Ditch. The Grand Canal has a bottom width of 20 feet, a depth of water of 3½ feet, and a fall of 2 feet to the mile, giving a velocity of 3 feet per second and a capacity of 210 second-feet. Its side slopes are 1 on 1½ and it is constructed about three-fourths in excavation. The dimensions of the Grand Canal are a little greater than those of the Consolidated, which latter is bifurcated just below its head into two main branches, the Maricopa and Salt River Valley Canals.

The main Arizona Canal has about 125 miles of laterals constructed, which command 77 000 acres of excellent arable land. The Consolidated and Grand Canals are together about 70 miles in length and have 75 miles of laterals, covering 73 000 additional acres of land. It is difficult to ascer-



FALL OR CROSS-CUT ON THE ARIZONA CANAL.

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tain the cost of the Consolidated properties, as they were constructed by various independent owners and by farmers. The main Arizona Canal, however, exclusive of these branches and all laterals, but including the dam and other works on its line, cost $580 000, the laterals $25 000 more, and the improvements to the present dam an additional sum of $25 000, making its total original cost about $630 000. As an indication of the value of land in this region when supplied with water for purposes of irrigation, it may be stated that water rights are sold to owners of land at the rate of about $1 000 for 80 acres, in addition to which an annual rental of $1.25 per acre irrigated is charged. On a portion of the line, water rentals are from $50 to $100 per second-foot.

As an example of the difficulties encountered and overcome in the present and last stage of irrigation development in this country, a brief description will be given of the canals of the Wyoming Development Company. These canals are now under construction and are partly completed. They cannot be considered good examples of the latest engineering practice, as no engineer is employed in building them, though originally designed by one. These canals, however, show the ingenious methods employed and the distances overcome in transporting water to valuable irrigable lands.

The canal is diverted from the Big Laramie River at a point about 120 miles north of Cheyenne and commands approximately 60 000 acres of irrigable lands between Sybille and Chugwater Creeks. At the point of diversion the Big Laramie has a maximum discharge in times of flood of about 8 000 second-feet, while the minimum discharge is as low as 100 second-feet at the lowest stage of summer flow. The water is diverted from the Laramie by means of a crib and stone dam 150 feet in length, resting on the coarse bowlder and gravel bed of the river. The maximum height of the weir is 4 feet, its top width being 16 feet, and it has a 4-foot vertical drop on the down stream side to an apron 16 feet in length composed of logs laid parallel to the course of the stream. The canal heads at the south end of the weir, and is regulated by a series of six wooden gates, each 5 feet wide and 9 feet in height, and operated by lever and ratchet. The first 2 000 feet of the canal consists of an open canal and approach cut in gravel, having a 30-foot bed-width, and capable of carrying 8 feet depth of water. Its grade is 7 feet per mile, and its side slopes ½ to 1, the discharge capacity being 650 second-feet. This

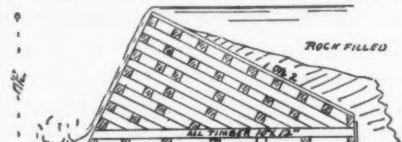
open cut terminates in a tunnel carried through a portion of the divide which separates the Big Laramie from Blue Grass River and tailing into a short branch of the latter stream. The tunnel is in rock, unlined, with an approximate cross-section of 7 x 8 feet, and is 31 feet in length with grade varying from 50 to 100 feet per mile. The terminus cut at the exit of the tunnel is in rock, and is 25 feet in maximum depth and 1 000 feet in length, the fall being 450 feet per mile and the bottom width 8 feet.

After being discharged into the Blue Grass Creek the water flows down this stream for a distance of 13 miles to its junction with the Sybille Creek, down which it flows a further distance of 1½ miles to a second diversion weir constructed across that stream. In Blue Grass Creek the total fall is 1 700 feet over a rock bottom and through a narrow and tortuous channel, and in Sybille Creek it falls an additional 30 feet. The diverting weir in Sybille Creek is constructed of wood founded on piling driven in the river gravels. The form of this weir is similar to the open weir described for the Del Norte Canal, and consists of eight bents 8 feet long across the stream and 8 feet in height, closed by flash-boards presenting a vertical face up-stream. The two banks of the river are protected by wooden planking for a distance of 12 feet up-stream and 18 feet down-stream. The second length of the canal heads above this weir, and is controlled by a set of regulating gates of the same dimensions and general form of construction as those described for the first section of the canal, but having a total width of 25 feet, disposed in five gates each 5 feet between centers, and built as a continuation of the diverting weir. This second section of the canal, which is the main distributing canal proper, is 34 miles in length, and is constructed for the first 20 miles of its course in gravel and hardpan, with much difficult side-hill work, some cuts being as deep as 40 feet. Its bed width is 25 feet, slopes 1 to 1, depth of water 5 feet and grade 2 feet per mile. In its course it encounters considerable side hill drainage, all of which is let into the canal by simple inlets, while waste-ways or escapes are constructed in the opposite bank of the canal and closed by simple flash-board regulators with discharging capacities according to circumstances, up to an extreme width of 30 feet. A few miles below the head of this main canal, an upper lateral is diverted which is controlled by a simple flash-board check built in the main canal. About 12 miles below the head of the main canal a second main distributing canal is diverted from Sybille

Creek by means of a weir similar to that just described, the height of which is 5 feet and its length 40 feet. This canal is 20 feet wide at bottom, and it has 4½ feet depth of water, slopes 1 to 1, and a grade of 2 feet per mile. It is 20 miles long and encounters some very heavy side-hill work in the first few miles.

A fine example of the present stage of canal development, and one which has been designed and constructed under the supervision of some of the best of our irrigating engineers, is the Bear River Canal in Utah, which is diverted from the Bear River at a point about 3½ miles above Colliston. This system consists essentially of a main western and main eastern canal, diverted one from either bank, while the former is divided into two principal branches, from which are taken the various laterals supplying the private ditches. At present the diversion weir and the first 6 to 8 miles of heavy rock work in the cañon are completed, and bring the water to the level of the broad valley lands which it is intended to irrigate. In all there are now completed 60 miles of main canals, with all flumes, regulators and other works, and a few miles of distributaries. When finally completed there will be 150 miles of main line and principal distributaries, and a sufficient mileage of laterals to enable the canal to irrigate 200 000 acres of excellent land.

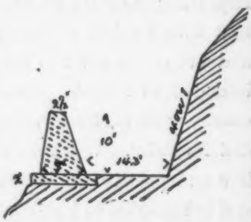
The average minimum discharge of Bear River at the weir site is about 1 000 second-feet, occurring in midsummer, while the maximum recorded flood discharge is less than 9 000 second-feet. The diversion weir, of which a cross-section is shown, is admirably located between high rock abutments, and with shallow rock foundation. It is constructed of crib work filled in with earth and loose stone, having a total height of 17½ feet, and a length at the bottom parallel to the channel of the stream of 38 feet, its length between abutments being 370 feet on the crest. The up-stream slope is 1 on 2 and the down-stream slope about 1 on ½, the water falling on a wooden apron, anchored to the bed rock. All timber is 10 x 12 inches, and is drift-bolted to the rock bed. The head-gates on the west side consist of five gates, each 4 feet wide by 7 feet high, of iron,



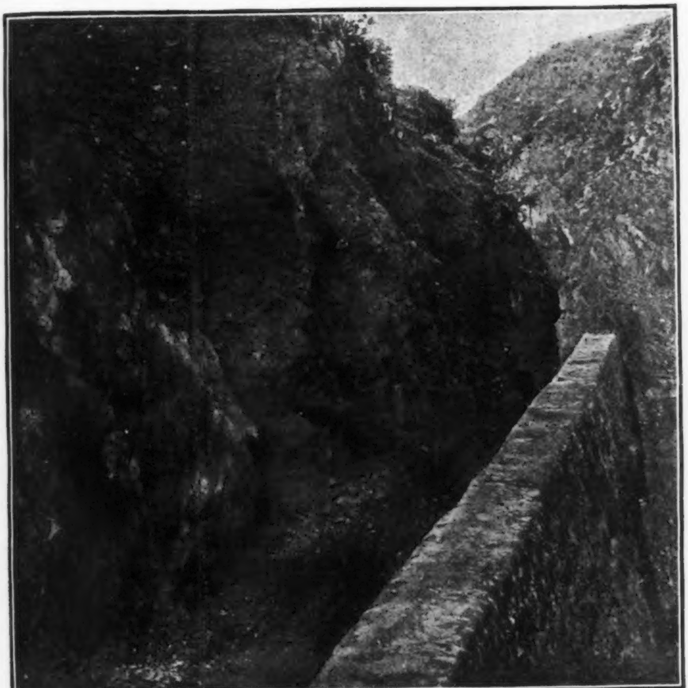
CROSS SECTION
BEAR RIVER WEIR

and built into substantial masonry-in-cement abutments and piers founded on rock. The head-gates to the eastern side canal are of essentially similar form of construction.

The first 2 miles of the east side canal are in heavy rock work, in which are two tunnels about 14 x 14 feet cross-section in homogeneous rock and unlined, the first being 423 feet long, and the second 200 feet in length. In this portion of the canal are a number of deep rock cuts (see Sketch and Plate XXXI), the greatest of which is 96 feet in height on the upper side, while in numerous places the lower side of the canal consists of a built up retaining wall of rubble-in-cement masonry, usually 10 feet in height inside, with a top width of 2½ feet, and a width at grade line of 7½ feet, as shown in the accompanying diagram. For 5 miles below the rock work the canal is excavated in steep earth hillsides, the slope of which is about 3 to 1. In this portion of the line are several deep fills across ravines, the beds of which are drained by means of drainage culverts carried through the bank. The greatest fill is 108 feet in depth at the center, and 500 feet long on grade, on the top of which is laid a wooden flume which carries the canal water and rests on long piling driven 60 feet into the fill. There are several such fills as this on both of the main canals, and two or three of them have already been washed out owing to leakage from the flumes or insufficient drainage way through the embankment. After reaching the level country the east side canal has a bottom width of 50 feet, a depth of water of 7 feet, with side slopes of 1 to 1 and a grade of 1 foot per mile. This canal as projected will have a length of 50 miles, terminating at the Ogden River, in the City of Ogden.

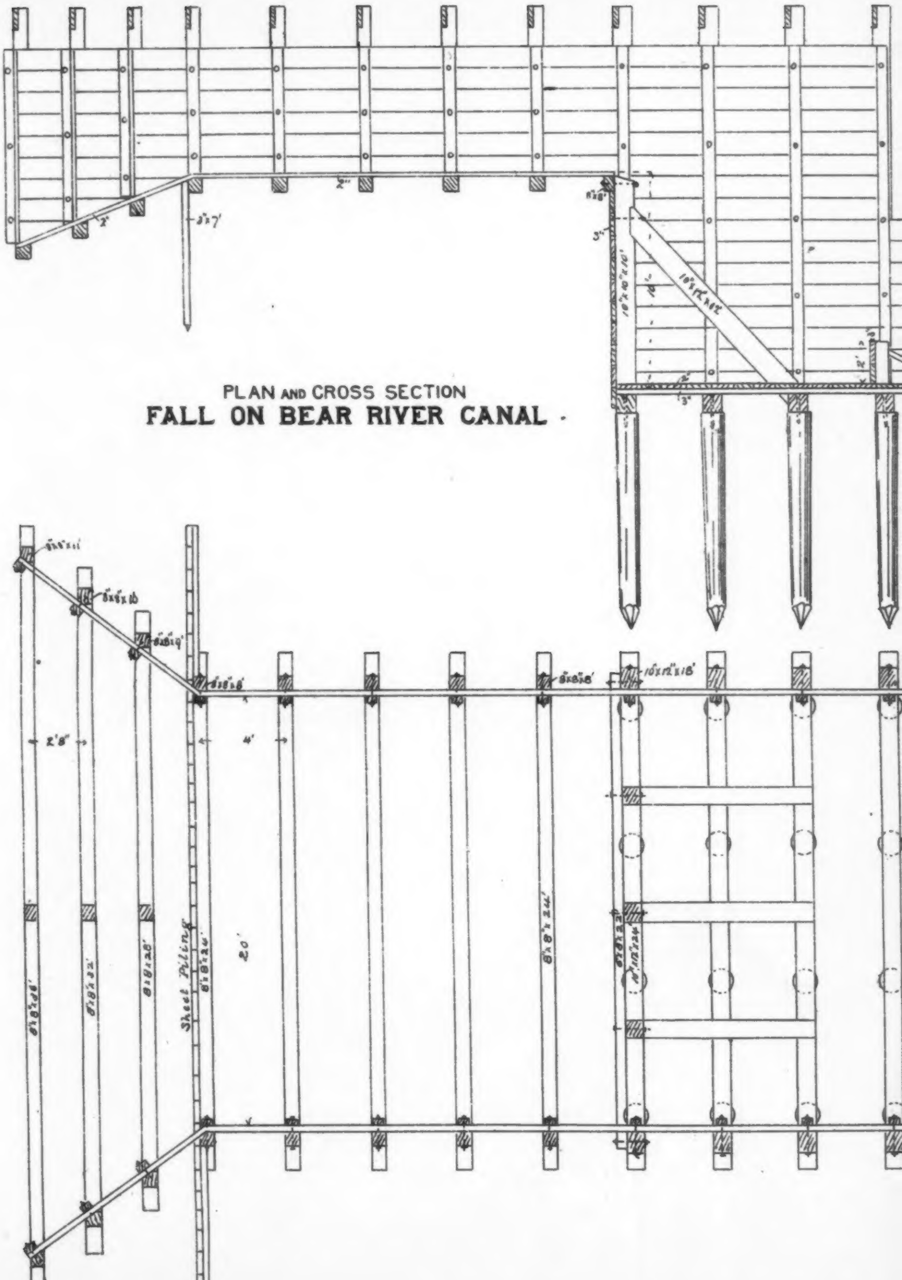


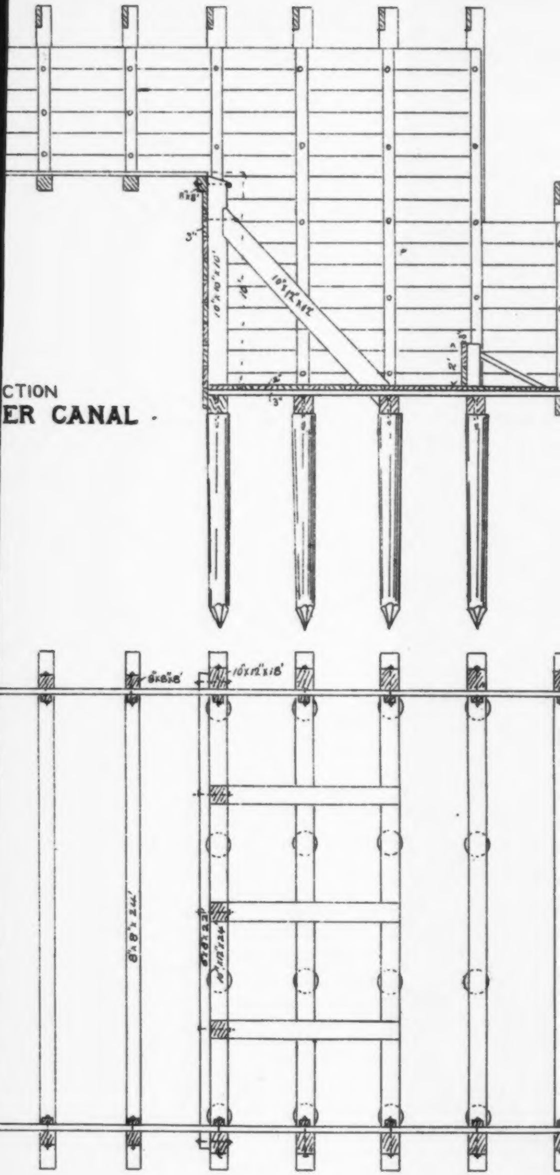
The west side canal for the first 9 000 feet of its length is likewise constructed in heavy rock cañon work. In this portion of its line are six tunnels, varying in length from 57 to 279 feet, and having the same cross-section and grade as the east side tunnels. Below and between the tunnels are eleven big retaining walls, of rubble masonry, having dimensions similar to those on the east side canal; while this portion of the canal has a bed width of 14.3 feet, a depth of 10 feet, and nearly vertical slopes through the rock work. Twelve hundred feet below the head of this canal is an escape gate, and 600 feet further down is a second



VIEW ON BEAR RIVER CANAL.

PLAN AND CROSS SECTION
FALL ON BEAR RIVER CANAL.





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ER CANAL .

PLATE XXXII.
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXV. No. 492.
WILSON ON
AMERICAN IRRIGATION.

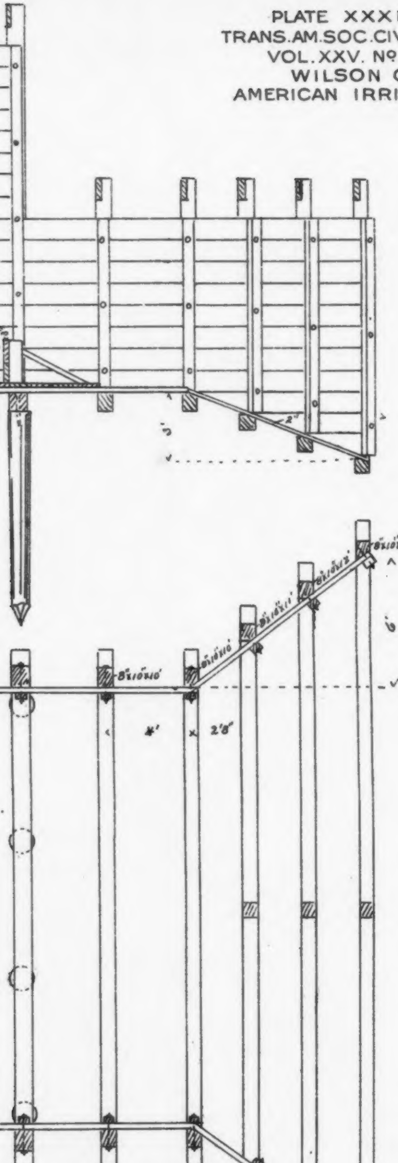


PLATE XXXIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXV. Nº 492.
WILSON ON
AMERICAN IRRIGATION.

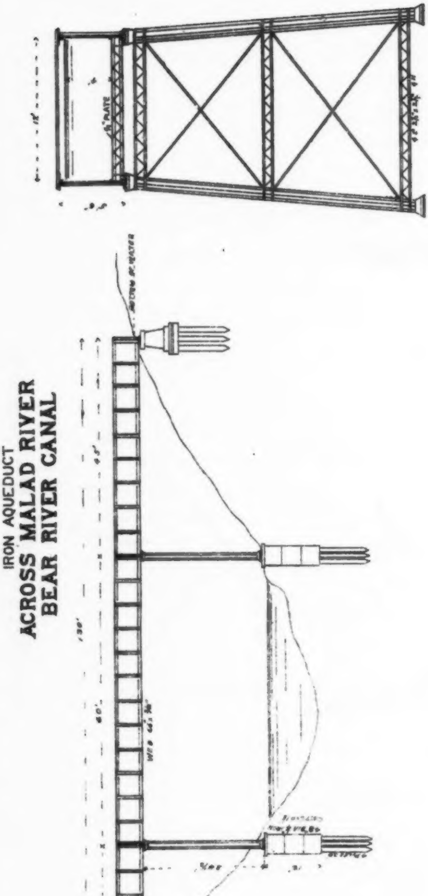
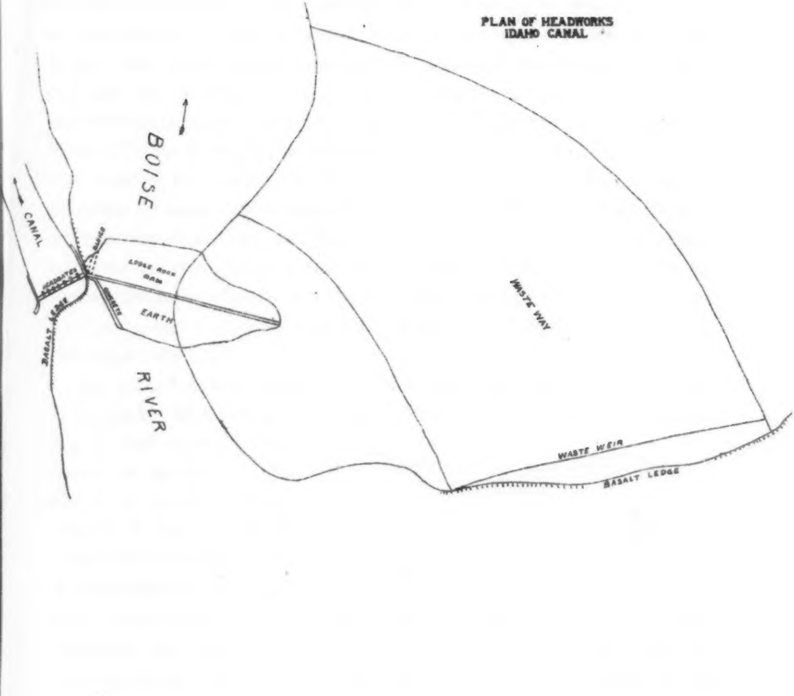
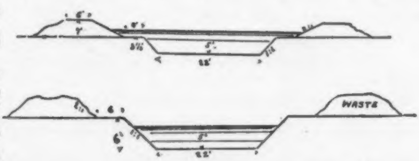


PLATE XXXIV.
TRANS. AM. SOC. CIV. ENG RS.
VOL. XXV. No 492.
WILSON ON
AMERICAN IRRIGATION.



escape gate, both discharging back into Bear River, and both having clear discharge openings of 12 feet in width, closed by three wooden gates, sliding between iron posts let into masonry piers. Below the second escape is a regulator closed by five gates, each 4 feet in width, by which the discharge in the canal itself can be controlled. After leaving the rock work, the canal enters steep hillside excavations in earth and clay similar to that on the east side, and extending to the sixth mile. In this side-hill work the bottom width of the canal is 14.3 feet, depth 12 feet, side slopes approximately 1 to 1, while there are several deep fills and cuts similar to those described for the east side. Beyond the ninth mile the canal crosses Malad River and valley on a high iron bridge and flume, 378 feet in length, and 80 feet in maximum height, supported on iron trestles, the river span of which is 70 feet. This bridge has approaches by means of wooden flumes which are 500 feet long, 20 feet wide in the clear, and carrying 7 feet in depth of water, the waterway over the iron bridge consisting of a similar wooden flume. In its course, the main western line has three falls, each of 7 feet in height, to lose grade, and it is projected for a length of 40 miles terminating at the Great Salt Lake.

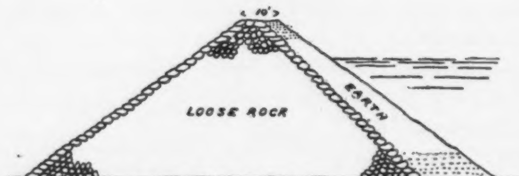
Six miles below the head of the West Side Canal the Corinne branch is diverted, which terminates in the twentieth mile of its course at the city of that name. This branch at its head is 22 feet wide at the bottom, carries 5 feet depth of water (see diagram), and is controlled at its head by a double set of regulating gates. In the main and Corinne branches, the bottom and sides of the canal are well protected by wooden aprons and wings. As this branch runs down the slope of the country it has in its course sixteen vertical falls varying from 4 to 12 feet in height, one of which is shown in Plate XXXII. At the fourteenth mile, Malad River is crossed on a high iron bridge founded on piles and iron cylinders filled with concrete. This bridge (see Plate XXXIII) consists of three principal bents from 25 to 60 feet in length, the peculiarity of its construction being that the superstructure forming the bridge itself is of iron plate girders and constitutes the flume which carries the water. The chief trouble with this iron flume is



the difficulty of keeping its junction with the wooden flume approaches in good condition. Among other works on the line of this canal are some inverted siphons or culverts of wood, carrying branch canals under the larger canals. One of these siphons consists of two tunnels of 8 x 8 timber, each having a clear waterway of $5\frac{3}{4}$ x $2\frac{1}{3}$ feet.

A magnificent illustration of the modern development of canal construction in the Western States is the canal of the Idaho Mining and Irrigation Company taken from the Boise River about 12 miles above the City of Boise, Idaho. This canal is interesting, not because it encounters any serious difficulties in construction, but because of the careful and intelligent thought which has been displayed by the engineer, Mr. A. D. Foote, in every detail of its design, and because of the magnitude of the canal and the permanent character of the various works constructed upon it. This canal is built with the two-fold object of irrigating about 350 000 acres of excellent agricultural land which is situated

CROSS SECTION
IDAHO WEIR



between the junctions of the Boise and Snake Rivers, and of supplying water for the hydraulic mining of extensive placer deposits on the lower banks of the Snake River, a short distance above its junction with the Boise. The climate and soil in this region are such that abundant crops of the more valuable grains, vegetables, and such fruits as apples, peaches, pears, grapes, etc., can be cultivated with the aid of irrigation. The maximum average discharge of the Boise River occurs between the months of April and July and is about 10 000 second-feet, increasing to the greatest known flood discharge of about 30 000 second-feet. Its minimum discharge is seldom less than 1 200 second-feet and the river may fall as low as this during the remaining months of the year. If necessary to supplement its discharge by artificial storage of its waters, several excellent reservoir sites have been discovered upon its headwaters, the combined capacity of which is nearly 225 000 acre-feet; while

the estimated cost of constructing these, averages about $1.50 per acre-foot, an unusually low figure.

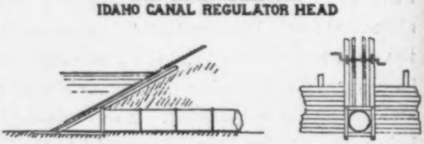
The diversion dam at the head of this canal is particularly well located, and as shown in the cross-section and Plate XXXIV consists of two parts, a main dam of loose rock with earth filling on the upper side crossing the channel of the Boise River, and over which it is not designed that the flood-waters shall pass; and a waste-way excavated through gravel on the north side of the river at a point about 345 feet above the dam, while on this waste-way is built a masonry weir over which flood-waters are discharged. The dam is 220 feet long on its crest and 43 feet in height above the river bed, with a top width of 10 feet, of which 3 feet of the inner side consists of the top of the earth filling, which at the bottom of the dam is 20 feet in width. The outside or rock slope is ½ to 1 and the up-stream or earth slope 1½ to 1. The whole is founded on the solid basalt which outcrops across the entire channel of the river at this point. Just above the dam a basalt ledge 12 feet in height borders the river bank, and on this ledge the waste-weir is constructed with a length of about 500 feet. The crest of this waste-weir is 8 feet below the crest of the dam, and the length of the waste-way is 200 feet from its entrance to its discharge point below the dam, while its capacity is estimated to be at least 30 000 second-feet. The cross-section of the weir is peculiar, being about 19 feet in width parallel to the length of the waste channel and 8 feet in maximum height; its upper slope has a batter of 6 to 1, while its lower slope has an easy ogee-shaped curve similar to that which the flowing water will assume.

The canal heads on the south bank of the river immediately adjacent to the end of the dam, in which latter is constructed an under or scouring sluice, the object of which is to prevent the deposition of silt in front of the head gates of the canal. These gates rest on the summit of a similar basalt ledge to that described as bordering the northern bank of the river and of equal height. The gates are constructed of the best rubble-in-cement masonry and are covered on top, forming an over-head bridge 19 feet in width. The regulating gates are 8 feet wide in the clear, and 19 feet in height to the crown of the semi-circular arch. The total height of these head-gates above the basalt ledge is 21 feet, and their height above the river bed 33 feet. The head-gates will be closed by means of roller curtains made of steel plates and angle iron for a height of 10 feet above the sill, above which the slats of the curtain are made of

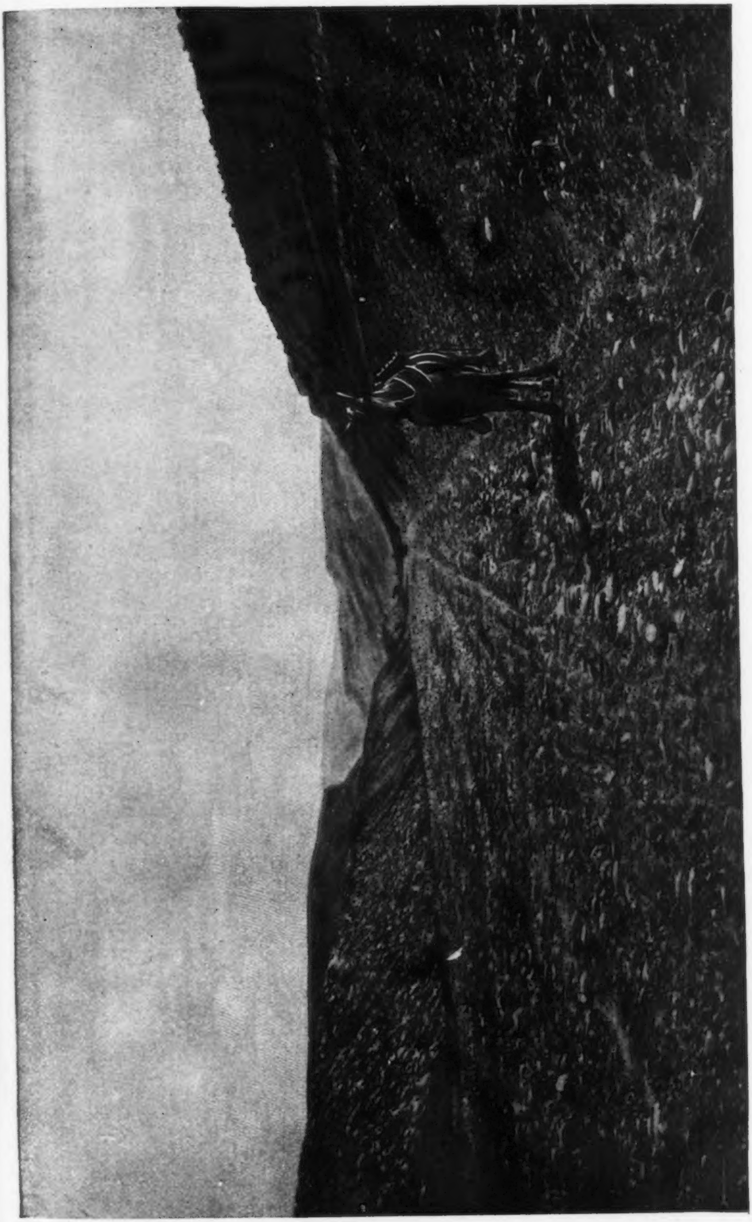
pine wood, each slat 8 inches wide. Each curtain is composed of twenty steel and eight wooden slats fastened to a central cast-iron roller at the bottom, and they are wound up by means of a single chain operated from a winch on the over-head bridge. These gates are modeled somewhat after those used on the open weirs on the River Seine, in France.

The first 2 miles of the main canal are excavated in loose lava rock in the steeply sloping sides of the cañon. There are no bad cuts nor fills encountered in this cañon work, and below it the canal encounters a couple of rather steep gravelly bluffs which its grade has to surmount, before it reaches the main body of the irrigable land. These lands are reached in about the eighth of a mile, after which the work is in simple level earth excavation, and encounters no other difficulties than three short stream crossings; one at the 15th mile, where Five Mile Creek is crossed by means of a short terreplein or raised embankment, with a wooden flume of about 24 feet in length founded on piles across the channel of the stream, and two other similar crossings at Ten Mile and Indian Creeks. In the first 8 miles of gravel bluff work below the cañon, the canal is constructed one-half in excavation and one-half in fill, the lower side being a high embankment which was constructed with scrapers and well tramped over. In one place this embankment is over 40 feet in height for a length of about 1 000 feet and is practically an earth dam for this distance. The capacity of the main canal (see Plate XXXV) is estimated at 2 585 second-feet; its bed width is 40 feet; height of bank, 12½ feet; depth of water 10 feet, and velocity of current, owing to its construction in the gravel and rock, as high as 5 feet per second. After getting above the bluffs to the level open country, all slopes to banks, both in and outside, are 1 on 1½, and a uniform grade of 2 feet per mile is given. The main canal has a total length of 70 miles, and tails into the Snake River at the placer mines above mentioned. At the 34th mile of its course the main eastern branch is diverted, which is 15 miles in length and likewise terminates in the Snake River.

CROSS SECTION AND ELEVATION
IDAHO CANAL REGULATOR HEAD



On the main line at the end of the cañon is an escape for the discharge of surplus water which tails back into the Boise River. This escape consists of a number of 48-inch cement pipes laid through



MAIN LINE OF THE IDAHO CANAL.

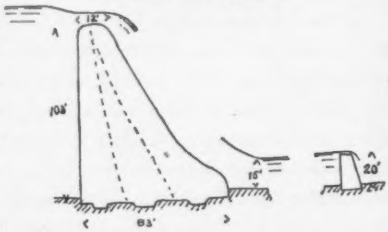
the canal bank and closed on the inner or water face by a sloping wooden gate laid at the angle of the canal banks and operated from above by rack and pinion. All distributary heads are constructed in the same manner of concrete pipes laid through the canal bank. In addition to the first escape below the cañon, provision is made for wasting water from the flumes crossing Five Mile, Ten Mile and Indian Creeks into those creeks. On the main line just below the bifurcation of the eastern branch is a fall 49 feet in height; about 6 miles further on is a 12-foot fall or drop, and further on, on the same line, are a 22-foot and 28-foot fall. These are not true falls, but are chutes or rapids constructed at such points as to take advantage of rock which crops out near the surface, and in which they are excavated, other portions of the rapids being masonry lined. All similar drops on the minor branches consist of inclined wooden flumes or rapids, laid on slopes of from 1 to 5 feet in 100.

By far the most interesting and magnificent canal under construction in this country is that of the Turlock Irrigation district on the west side of the San Joaquin Valley, in Central California. This canal is interesting as an example of the result and operation of the Wright Irrigation Law, recently passed in California, as well as from the magnificence of its works and the skill displayed by its engineer, Mr. E. H. Barton, in overcoming the difficulties encountered in locating the first few miles of its diverson line. The Turlock district is so outlined that all of the 176 000 acres included within its borders can be watered from the same source of supply. This district is bonded for $600 000, but it is estimated that before the works under construction are completed a further sum about equal to this will have to be raised by the issuance of additional bonds.

The Turlock Canal is diverted from the Tuolumne River at the mouth of the cañon. This river may attain a maximum flood discharge of 150 000 second-feet, while its minimum discharge rarely falls below 2 000 second-feet. The head-works of the canal consist of a masonry dam, which is constructed as a common diversion weir for the Turlock Canal and the canal of the Modesto Irrigation district, which heads on the opposite or north bank of the river. This weir is located between high cañon walls 2 miles above the town of La Grange, at a point where the abutments and foundation of the weir are on a firm homogeneous dioritic

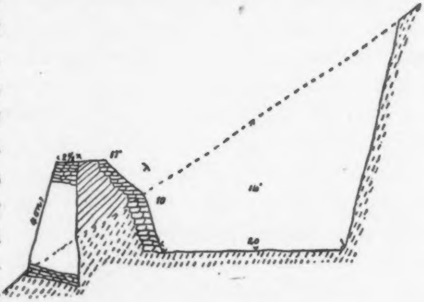
basalt in which scarcely any excavation will be required. The weir, over which the flood discharges of the river will pass, is constructed throughout of uncoursed rubble masonry in Portland cement, and has a length on its crest of 310 feet. Its maximum height, including foundations, is 103 feet, and the maximum height of the overfall of water is 98 feet, to a water-cushion formed by a subsidiary weir built of rubble masonry and located at a point about 200 feet below the main weir in a narrower portion of the cañon, as shown in the accompanying diagram. The main weir has a top width of 20 feet, and is designed on a cross-section somewhat similar to that of the Vyrnwy Dam for the Liverpool water supply; its upper face being nearly vertical, while its lower face is given a curve similar to that which the water flowing over its face will assume. This weir is so designed that the maximum pressure on the toe, when full, will be 6.3 tons per square foot, which is well within the limits which the material will stand. The subsidiary weir is 120 feet long on top, 12 feet in width and 20 feet in maximum height, and will back the water up to a depth of 15 feet on the toe of the upper or main weir giving a water-cushion of that depth into which the flood-waters will fall, thus to a certain extent diminishing their shock.

CROSS SECTION
TURLOCK WEIR



The arrangement of the head-gates of the canal is peculiar. The canal is diverted from the south bank of the cañon at a point about 50 feet above the end of the main weir. Owing to the great floods which occur in this narrow cañon, the water may rise as much as 15 feet in an hour, and the maximum height which it is estimated to reach above the sill of the canal is 16 feet. The pressure of this height of water on the regulator head would be so great as to greatly increase the cost of its construction, accordingly the canal heads immediately in a tunnel 560 feet in length, blasted through the rock of the cañon walls and having no regulating apparatus at its entrance. Where it discharges into the open cut, which is the commencement of the canal, regulating gates and scouring or escape sluices are placed. The entrance tunnel is 12 feet wide at the bottom, 5 feet in height to the springing of the arch, above which it is

semi-circular with a 6-foot radius. Its slope is 24 feet per mile, and it is excavated in a firm dioritic rock which requires no lining. The regulators in the canal head below the exit of the tunnel consist of six gates, each 3 feet wide in the clear and 12 feet in height. These gates are constructed of timber and iron, and slide on angle iron bearings firmly let into the rock and set in concrete. The escape is set at right angles to the canal line, heading immediately above the regulator between it and the end of the tunnel, and tailing back into the Tuolumne River a short distance below the subsidiary weir. Like the regulator, the escape consists of six gates, each 3 feet wide in the clear, 12 feet high, and constructed of similar material and in like manner. It is estimated that, whereas a maximum flood of 16 feet over the sill of the tunnel will give a discharge in front of the regulator and escape of about 4 000 second-feet with a velocity of 20 feet per second, the wasting capacity of the escape will be at least 6 000 second-feet, thus fully insuring the canal against accident from this source.



Below the regulating gates the main canal proper begins, having a capacity of 1 500 second-feet. For the first 6 200 feet it is excavated in slate rock on a steep hill-side. It has a bed width of 20 feet, depth of water of 10 feet, and an upper rock slope of $\frac{1}{2}$ to 1; while the lower bank or down-hill slope, where gullies are crossed, is, as shown in the accompanying diagram, built up with an inner slope of $\frac{1}{2}$ to 1, and faced with 18 inches of dry laid retaining wall inside and outside, the interior of the bank consisting of a well puddled earth core 12 feet in width. Where this portion of the canal is on ordinary sloping ground, not crossing gulches, its dimensions are the same, but the inner face only has the 18 inches of rip-rapping, and the down-hill slope of the bank consists of dirt and other spoil, the top width of the bank in such places being 5 feet, and the puddle wall 5 feet in thickness. This portion of the canal line has a grade of 7.92 feet per mile, which gives a velocity of $7\frac{1}{3}$ feet per second.

At the end of this slate-rock work the canal empties into Snake

Ravine, just above its junction with the Tuolumne River, up which ravine the water of the canal runs for 940 feet. This is effected by constructing an earth dam across the mouth of the ravine just below the entrance of the canal, which raises the surface of the water so as to form a small settling reservoir, and produces a flow up the course of the ravine for the distance above mentioned. The earth dam is 20 feet wide on top, 318 feet long on the crest, has slopes of 2 to 1 and a maximum height of 52 feet. This dam was partly constructed of material borrowed from its abutments and the canal excavation, and partly by a silting process from material washed out of a hydraulic cut at the upper end of the ravine. This hydraulic cut is 800 feet in length and 45 feet in maximum height, with slopes of 1 to 1, and a grade of 5 feet per mile. Owing to the abundance of water procurable, this cut was more cheaply excavated by the hydraulic mining process than it could have been by other means. At the far end of this cut the canal enters an old hydraulic wash, in which its waters spread out, and which is utilized as its channel for a length of 2 380 feet, after which it enters a rock cut 860 feet long, with a maximum depth of 45 feet and a similar cross-section to the cut first described.

At the end of this rock-cut the canal water is discharged into Dry Creek, down which it flows for a distance of 6 500 feet on a grade of 12 feet to the mile, and from which it is diverted by means of an earth dam 460 feet long. This dam has a maximum height of 23 feet with both slopes of 3 to 1, and is rip-rapped to a depth of 3 feet on its upper face. At the south end of the dam it abuts on sandstone rock, in which a waste-way is cut 50 feet wide, with its sill 4 feet below the crest of the dam, and which will discharge back into the creek 180 feet below the toe of the dam. Between the waste-way and the end of the dam is a waste-gate which it is intended shall be used in times of freshets, for Dry Creek has a maximum discharge of 4 000 second-feet, and as the freshets are quick and violent a large wasting capacity is necessary. These waste-gates are ten in number, each 3 feet wide in the clear and 10 feet in depth. They fall automatically outward or down-stream, being hinged at the bottom to a concrete floor laid on the bed-rock, and when raised they are attached by chains to the piers.

From Dry Creek the canal is excavated for about a mile in heavy, sandy loam, in which it has a bed width of 30 feet, with slopes, 2 to 1, a depth of 10 feet, and a grade of 1½ feet per mile. At the end of this

PLATE XXXVI.
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WILSON ON AMERICAN IRRIGATION.

TUNNEL ON DRY CREEK.

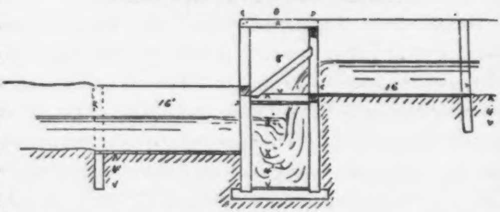
excavation the canal crosses Dry Creek on a flume 62 feet in height and 450 feet long, after crossing which it enters a series of three tunnels, the cross-sections of which are nearly similar to that of the first tunnel described, while they are excavated in a tufa and sandstone which will require no timbering. The first tunnel (see Plate XXXVI) is 211 feet in length, the second 400 feet, and the third 400 feet in length, while they are separated by short, open cuts, excavated in hardpan and clay, which are respectively 250 and 300 feet in length. The last tunnel discharges into Delaney Gulch, which is crossed directly by constructing a high bank or earth dam below the canal, the total length of which is 180 feet, its maximum height being 40 feet, and its top width 20 feet. The volume of discharge of this gulch is so trifling that it was unnecessary to provide a waste-way or escape at this point. Immediately after crossing the gulch the canal enters a cut 8 feet in maximum depth, with the same cross-section and grade as the first cut, and having a length of 3 300 feet. The canal is now widened to a bed width of 35 feet and depth of 10 feet, and is given a grade of 1 foot per mile. At the end of a mile and a half, Peasley Creek is crossed on a trestle and flume 60 feet in height and 360 feet long, the water-way on which is 20 feet wide and 7 feet in depth. This flume is provided with an escape constructed in its bottom and discharging into two small sloping flumes which lead the water down into the bed of Peasley Creek.

At the end of the flume the main canal is reached and traversed for a distance of 11 miles, in which course are two rock cuts, each 3 000 feet long, and respectively 20 and 30 feet in maximum height. In these cuts the cross-section is 35 feet wide on bottom, depth of water 7½ feet, and grade 5 feet per mile. The remainder of this length of the canal varies in cross-section according to soil, but the majority of it has a bottom width of 70 feet, and depth of water of 7½ feet, slopes 2 to 1, and a grade of 1 foot per mile. Previous to choosing this location a careful detailed topographic survey was made of the entire Turlock district at a scale of 1 000 feet to the inch and in 5 foot contours, and from this map the location was chosen. This is so designed as to take advantage of several natural channels consisting of the creeks and gulches mentioned, of an old hydraulic excavation, and of the opportunities afforded for excavating two deep cuts by the hydraulic process. To be sure, one slight disadvantage may be claimed, because of the large water-surface exposed to evaporation by the creation of the various small reservoirs. This,

however, is not a defect, for the water supply will exceed the demands for a number of years to come, and these basins act as settling reservoirs in which the turbid water of the river will be freed of much of the sediment which it carries in suspension. Before the water becomes scarce, these basins will be filled full by sedimentation, and in the flats thus created a canal channel of proper cross-section can be maintained. It is expected that in the course of a few years the river bed above the main diversion weir will be silted up to the level of its crest; there are no objections to this, as it is not intended to utilize the basin above the weir as a storage reservoir.

The main canal as outlined above consists for the 18 miles of its length of a purely diversion channel, the object of which is to bring the water to the irrigable lands included within the area of the Turlock district. At the terminus of this diversion line the canal will at once begin to do duty by watering the lands, and below this point the main line is divided into four main branches, each of which has a bottom width of 30 feet, depth of water of 5 feet, and grade of 2 feet per mile, their aggregate length being 80 miles. In addition to these main branches minor distributaries leading the water to each section of land and having a total length of 180 miles are being constructed. The lower discharge branches and distributaries are so designed as to give a uniform velocity of 2½ feet per second, in order that any matter carried in suspension will be held up until deposited on the agricultural lands instead of in the canals.

CROSS SECTION
FALL ON TURLOCK CANAL



It is interesting to note that while the contract prices for excavating the two deep cuts by ordinary processes was forty-eight cents per cubic yard, they were excavated by the hydraulic process for thirty-one cents per yard, the material being moved one-half mile, while gold to the value of $13 000 was saved in the washing, thus reducing the cost of excavation by four cents per yard. In the course of the main canal nine falls are inserted in order to reduce the grade. These vary in height

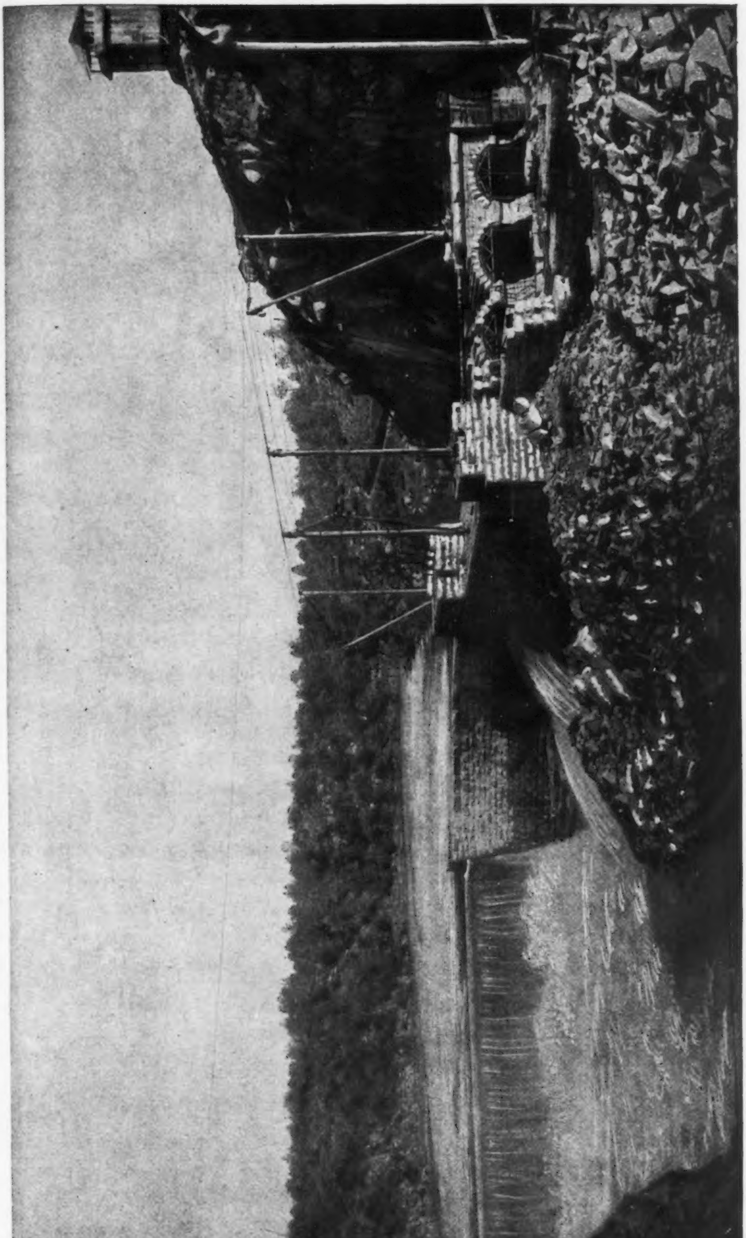
from 4 to 11 feet, while the bed width of the channel is contracted from 70 feet to a clear width of 40 feet through the falls, the general form of construction of which is indicated in the accompanying illustration. Falls 5 feet in height drop into water cushions 4 feet in depth, the 11-foot falls having water cushions 6 feet in depth, the object of which is to break the force of the fall. Across the channel the falls are divided into four bays, each 10 feet wide, by means of vertical rows of plank inserted in the lower portion of the fall. These diaphragms are intended to keep the current straight after passing the fall, thus preventing back currents and scouring, in which effect they are aided by the widening of the channel again by means of wings from 40 to 70 feet. It was calculated by the weir formula that by narrowing the channel above the fall from 70 to 40 feet the lowering of the depth of the water would be prevented with its consequent increase of velocity and scour.

The works of the Folsom Water Power Company, situated on both banks of the American River, near Folsom, Cal., are the most substantial and elaborate of their kind that have been constructed either in this country, Europe, or India, so far as the writer's experience indicates. These works are built with the object of furnishing water power to the State Prison at Folsom, and for manufacturing and milling purposes in the City of Folsom, after which the water used in producing the power passes on to the irrigable lands which the canals will develop in the near future. The chief feature of interest in connection with these works as at present constructed is the massive and substantial character of the diversion weir which is situated on the American River, 1½ miles above the town of Folsom, and of the main canal which is completed as far as Folsom. The waters carried in the main east side canal after they have been utilized at the second fall in Folsom for the development of water power, can be readily transported in simply constructed branches and laterals to 125 000 acres of excellent irrigable lands in Sacramento County, and a ready sale will be found for all the water which can be made available for this purpose. One of the peculiarities of this project is the arrangement by which it is constructed in co-operation with the State Prison, which is situated near the site of the dam a mile above Folsom. In consideration of the Water Power Company giving the prison authorities about 400 acres of quarry land adjacent to the prison, a right of way through the company's property for a railway, and a 7-foot head of fall

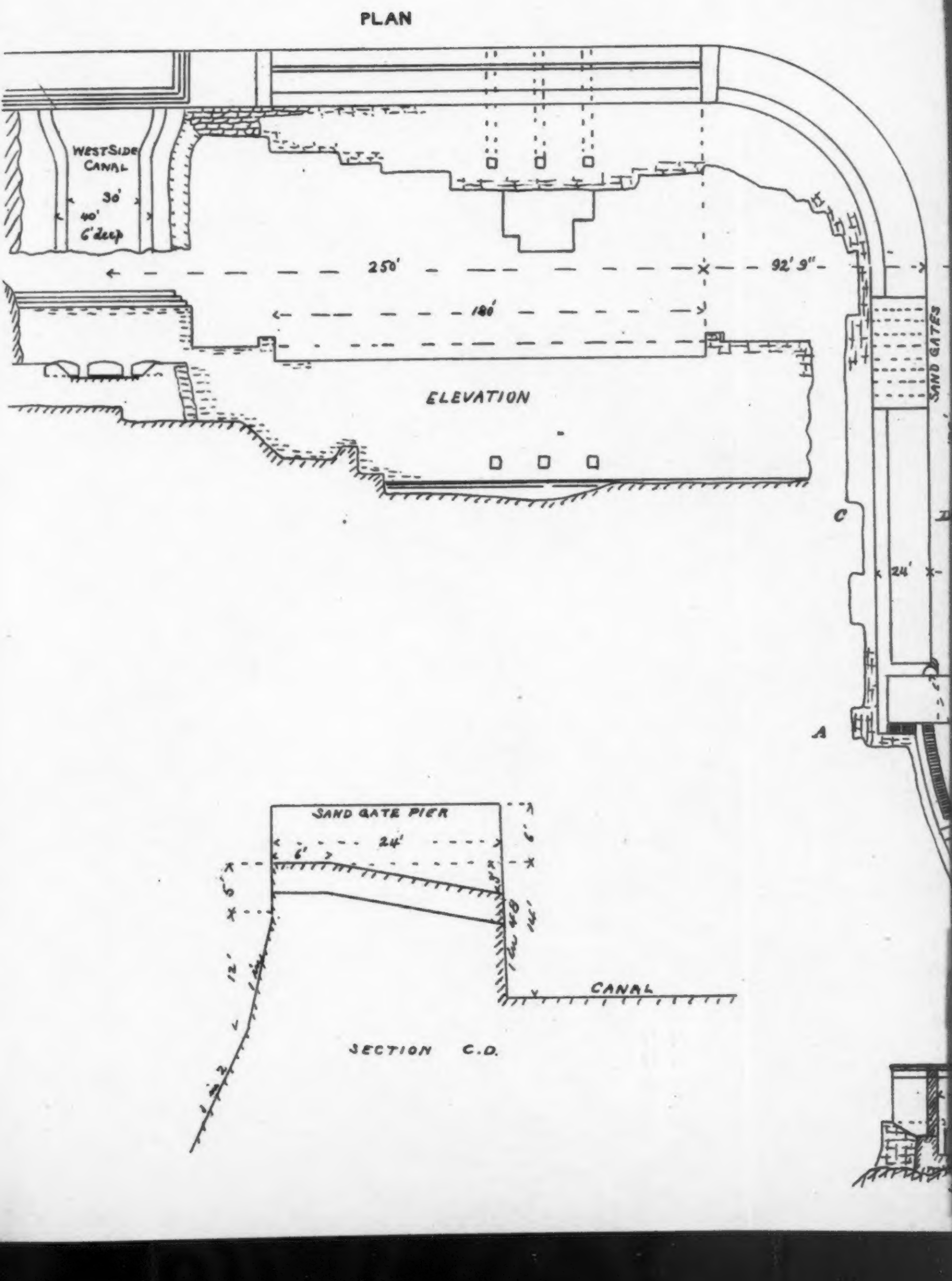
in the prison yard for the generation of water power from the entire discharge of the canal, the prison authorities have given to the company all the labor required in the construction of this work, this labor being performed by the convicts in the prison.

The great diversion weir (see Plate XXXVII and XXXVIII) was designed and built under the direction of Mr. P. A. Humbert, chief engineer. This work is constructed of the best coursed granite rubble-in-cement, founded on a firm bedrock and abutting against granite walls. The main tangent of the weir across the channel of the American River is 250 feet in length, while the first 250 feet of the canal on the left bank of the river is constructed as a prolongation of the weir, thus giving 500 feet in length of discharge over which the floods of the American River can fall. The flood discharges of the stream are very violent, and it is anticipated that the maximum flood passing over the crest of the weir will be about 34 feet in depth, a depth of 31 feet having already been gauged. As will be observed from the diagram, the cross-section of the weir is very substantial; it is 24 feet in width on top, 87 feet wide at the bottom, with a massive buttress extending well down below the toe, the object of which is to counteract the scouring effect of the water. It is 69½ feet in height on the up-stream face, its total height being 98 feet. In the center of the weir, and on a level with the low water surface of the river, is a set of three under or scouring sluices, each 4 x 4 feet, which discharge the silt-laden flood waters of the river under a head of 60 feet. While these under-sluices have not impaired the integrity of the structure, they have been of little service in preventing the deposit of silt, as their area compared with that of the flood is relatively small.

One hundred and eighty feet in length of the center of the weir is lowered to a depth of 6 feet below that of the remainder of the crest, and forms a discharge weir which is closed by a shutter the height of the opening. As this river carries in suspension a great amount of sediment, it was expected that without some flushing apparatus it would soon silt up on a level with the head-gates, accordingly in order that the silt deposit in this place may be kept down to a level with the sills of the head-gates, this waste-way has been introduced. When the shutter closing the waste-way is lowered the water will carry out the sediment down to a level with the crest of the waste-way, and when raised it will divert the water into the head of the canal. This shutter is of peculiar construction:



HEAD-WORKS AND WEIRS ACROSS AMERICAN RIVER, FOLSOM CANAL, CALIFORNIA.


PLAN
WEST SIDE CANAL
30'
40'
6' deep
250'
92' 9"
180'
SAND GATES
ELEVATION
C
D
24'
A
SAND GATE PIER
24'
6'
6'
5'
3'
12'
14'
CANAL
SECTION C.D.

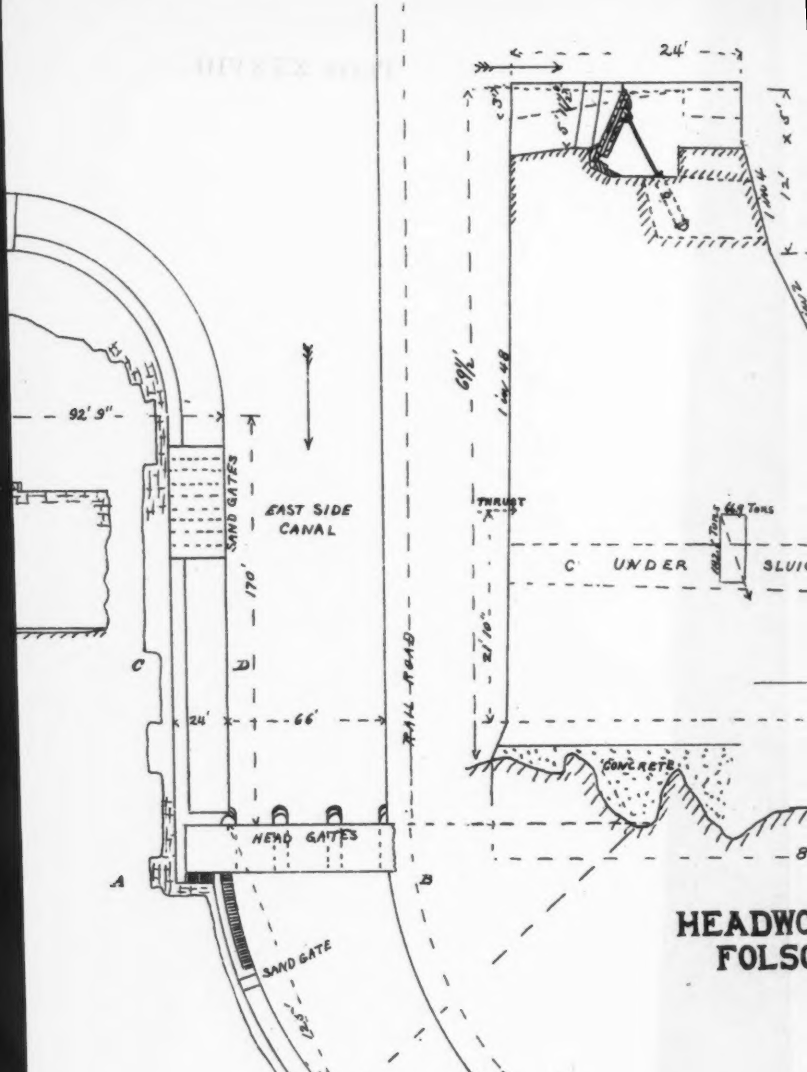

24'
69½'
1 in 48
THRUST
44.9 Tons
C UNDER SLUI
21'10"
CONCRETE
92' 9"
EAST SIDE
CANAL
SAND GATES
170'
C
D
24'
66'
RAIL ROAD
HEAD GATES
A
B
SAND GATE
50'
66'
16'
SECTION AB
HEADWO
FOLSO

PLATE XXXVIII.
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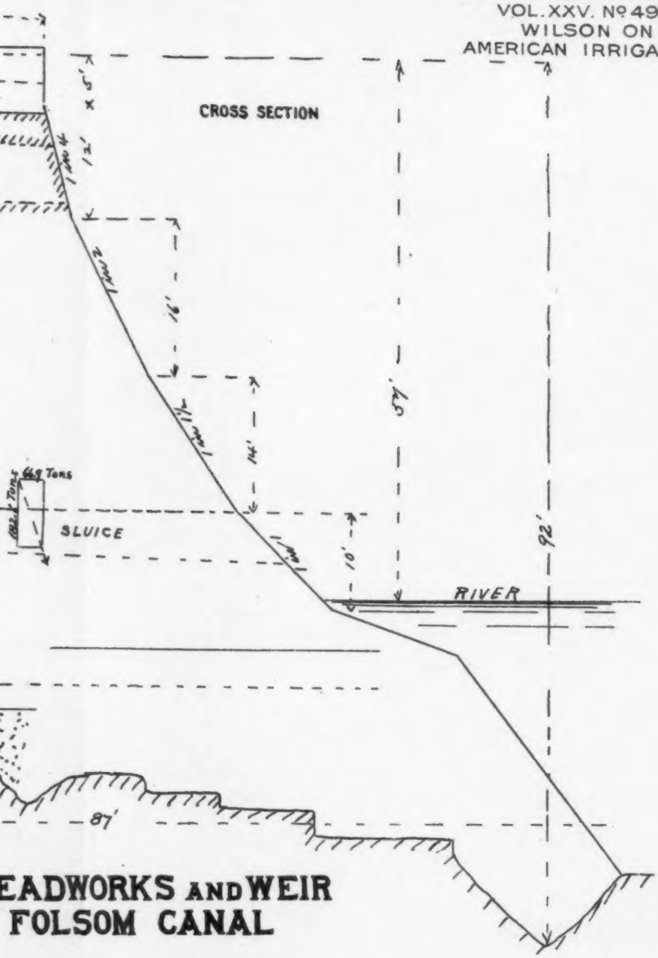


EADWORKS AND WEIR
FOLSOM CANAL

it consists of a single Pratt iron truss the full length of the opening and of equal depth. This shutter is opened and closed by means of a row of hydraulic pistons which are operated from an accumulator placed near the power-house which is at the State Prison. It is thus possible, under pressure of the greatest head, to manipulate this shutter with ease. In like manner the gates closing the head regulator, the sand-gates and other similar works, are all operated by hydraulic power from the same accumulator.

Heading from both banks of the river, at either end of the weir, are a west and east side canal. The construction of that on the west side has not yet been commenced, and will not probably be undertaken for several years, until the irrigation resources on the other side have been more fully developed. Immediately in front of and above the head of the east side canal are a set of four under-sluice gates placed 6 feet below the grade of the canal and discharging back directly into the river. These gates, like the regulating head and all the first portion of the canal line, are constructed, as is the weir, of massive granite masonry. Each of these under-sluice gates is 5 x 6 feet in the clear, and opposite and below them is a sub-grade in the canal-bed across its entire width, in which sediment will be collected and from which it will be sluiced out by opening the gates. The head-gates themselves are three in number, each 16 feet in width and 12 feet high, and are of wood heavily trussed with iron and sliding between granite piers and abutments. Below the head-gates the first mile and a half of the canal, as far as the town of Folsom, is excavated on a steeply sloping granite hillside. This canal is 8 feet deep, 34 feet wide at the bottom, 50 feet wide at top, and is given a grade .0005 in 1, which produces a velocity of 3.6 feet per second and a discharge capacity of 1 210 second-feet. The banks of this canal and the dry rubble outer wall are given a slope of 1 on 1, the slope of the country rock above the maximum water line of the canal being ½ on 1 and the top width of the lower retaining wall 10 feet. In the first 1 700 feet of the canal are seven separate sand-gates, each 5 feet wide by 10 feet high, and similar in construction to the scouring sluices and other works. These are of masonry set in the loose rubble bank on the lower side of the canal and discharge into the river. Their object is to clear the water as far as possible from sediment before it is passed through the turbines.

COMBINED STORAGE WORKS.

The storage of water for the purpose of supplementing the flow of intermittent streams, and thus increasing the amount of water available for irrigation during a certain short period of time, has been practiced as long in this country as has irrigation from perennial streams. The general principle of water storage for purposes of irrigation is so similar to that of storing it for domestic use that it seems to require little discussion in connection with an article of this kind. At the same time this subject has created so much discussion and comment in the last few years among legislators, capitalists, and engineers, that it is desirable to describe the methods employed and give details of some of the more important works completed or under construction in this country.

Since the passage by Congress of the Act of October 2d, 1888, providing for the survey by Government engineers and the withdrawal from occupation of lands included within storage reservoir sites, the growth of popular interest in the subject of water storage has received a marked impetus, and has led to the development of a great number of storage projects, a few of which are now under construction. Many of these are within themselves excellent and feasible projects and will some day be undertaken, while a large proportion will necessarily in the course of time prove impracticable and be abandoned. Connected with this bill were certain laws instructing the Director of the United States Geological Survey to outline irrigation projects and to designate for withdrawal from sale or occupancy such Government lands as might be reclaimed by future irrigation enterprises. For nearly two years active and important work was conducted to this end, both in the designation of irrigable land and reservoir sites, and in the survey and designing of many important canal and reservoir projects which will at some future day be constructed. The unfortunate clause directing the withdrawal of irrigable lands from sale or occupancy created the most violent opposition to this law, which, while it came from the west, was instigated by eastern capitalists interested in cattle raising or land speculation, who were thus deprived of the opportunity to acquire land with which to carry on their business. Much of the opposition to this law arose also from a misinterpretation of the effect which it would have on the acquirement of land by irrigation companies, and the perpetuation of this mistake by interested parties. Some of it came also from shortsighted merchants and real estate agents in western towns, who, while

they feared that the closing of agricultural lands to settlement for a short period of years would retard immigration and injure the present growth of the west, failed to appreciate the great benefits which must ultimately accrue from such legislation, in the more healthy and permanent growth which must result from intelligent irrigation development and legislation.

One good effect of this first law was, that though it was repealed, it has resulted in marked modifications and improvements of the land laws as they previously existed, reducing the amount of land which may be acquired by a single settler from 1 120 to 320 acres, and causing the repeal of the Pre-emption and Timber Culture Acts, and a marked reduction in the amount of land which may be procured under the Desert Act, as well as a decided improvement in the wording of that act itself. If the Government agitation on the irrigation subject had gone no further than this, it would have done a great good. It has, however, done much more, for it has resulted in the inauguration of a series of hydraulic investigations which have largely increased our knowledge of the evaporation, run-off from catchment basins, and the discharge of many of the larger streams of the arid region; and it has resulted above all in giving an added impetus to the construction of contour topographical maps in that region on a larger scale than that on which they were previously made, thus facilitating the discovery and projection of storage reservoir and canal enterprises, and adding largely to our knowledge of the feasibility of these, by showing the areas and slopes of the catchment basins of the different streams on which they are located. Above all, however, the retention of one of the most important provisions of the original bill, that which authorizes the survey and withdrawal from occupation of storage reservoir sites, lays the groundwork in the future for the development of many valuable water-storage projects; for it prevents agricultural settlers, and above all cattle and land speculators, from grabbing these sites, and then holding them back that they may charge *bona fide* irrigation companies such exorbitant prices for them as to either impede or prevent the development of their operations.

A notable instance of the damage done by such land speculators, is that of the parties who occupied some of the lands which would be flooded by the Sweetwater dam in San Diego County, Cal. These individuals kept very quiet regarding their operations and intentions during

the construction of that work, but on its completion they brought an injunction against the irrigation company to prevent their permitting the water to rise high enough to flood back on their lands, and asked such an exorbitant price per acre for what, until the construction of that reservoir, was barren arid land, as to make its purchase almost prohibitive. The case dragged for several years through the various courts, and each time a decision was rendered in favor of the land-holders they raised the price of their property, in consequence of which the Sweetwater reservoir was never more than half full until within the last few months, when a compromise was effected between the contending parties.

In such an article as this, space, and the subject chosen, will not permit of a detailed discussion of the laws governing the discharge from catchment basins, the subject of evaporation, the considerations governing the location and the selection of reservoir and dam sites, nor the innumerable hydraulic problems connected with the designing of storage projects. Accordingly, as was done with the subject of canals, this portion of the paper will be confined to a description of the engineering details of various important storage projects completed or under construction for purposes of irrigation, taking these projects up progressively in the order of their importance and excellence of design, while dividing them among the two principal heads of earthen and masonry dams.

On the comparatively level bench-lands in Colorado, which slope away to the eastward from the foot of the Rocky Mountains, are a number of natural depressions which have been converted by the excavation of deep cuts or the construction of embankments into small storage reservoirs, into which are discharged the waters from the canal lines on which they may be located, and in which these waters are stored until wanted for irrigating the lands situated below them. Several reservoirs of this type have been designed and constructed in other parts of the west, notably, one on the line of the Florence Canal in the Gila Valley in southern Arizona, one on the line of the projected Sun River Canal in northern Montana, and a few others in other localities. It is not necessary to describe these works in detail, their construction is of the simplest, while the general plan on which they are projected is not the most commendable. It is not desirable to store water in the lower and more open plains country, nor to make use for purposes of storage of

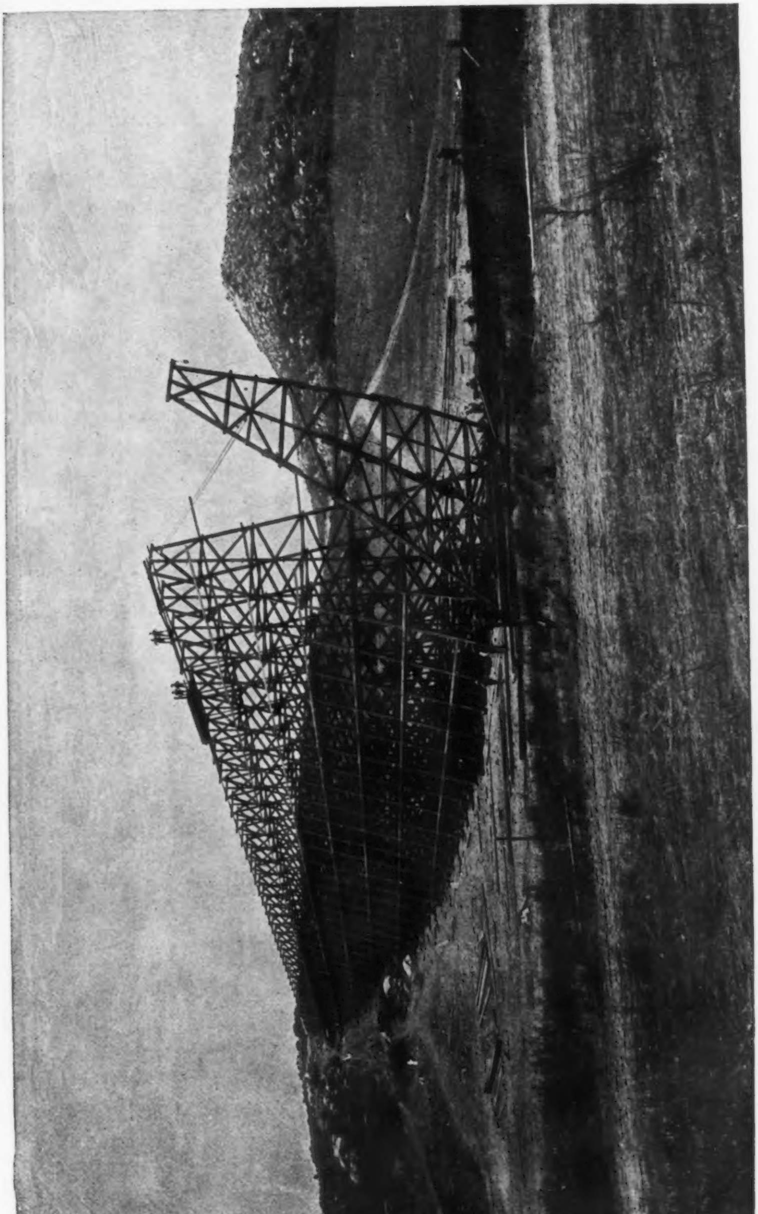
natural basins or depressions, because the loss by evaporation at the lower altitudes is great, and because the discharge capacity of the canal from which these basins are filled must be made considerably greater than would be necessary if they were merely used as direct irrigation canals. Storage reservoirs of the other class, those which are located at higher altitudes and among the more rugged mountainous regions, have several advantages. The loss by evaporation from their surfaces is reduced to a minimum. The water which is stored in them is comparatively free of sediment and their life is correspondingly increased. All the water which runs off their catchment basins flows immediately into these reservoirs without the interposition of any artificial channel, and it can accordingly all be stored, providing the capacity of the reservoir is sufficiently great. And above all, sites for these reservoirs can usually be found where, by the construction of short and high dams the water surface exposed both to evaporation and absorption is reduced to a minimum.

One of the first reservoirs of this type constructed in this country, which furnishes a good type of earthen dam and combined natural and artificial distributing system, is the Cuyamaca reservoir, situated in the Coast Range, about 70 miles east of San Diego, Cal., at an altitude of 5 500 feet. This work is the property of the San Diego Flume Company which was organized in 1886, with the object of bringing water from the high Coast Range mountains for the supply of San Diego City with domestic water, and to furnish the ranches in its neighborhood with a sufficient water supply for purposes of irrigation. The system consists of a storage reservoir having a capacity of 830 acre-feet, from which water is turned back into Bowlder Creek, down which it flows for a distance of 12¼ miles with a fall of 4 000 feet to the main diverting dam, the altitude of which is 800 feet, and above which the diversion canal heads. Owing to the value and scarcity of water in this neighborhood the canal is not in open excavation, as the loss by evaporation and absorption in such a channel would be too great, but it consists of a wooden flume 36 miles in length, built for a majority of that distance along the steeply sloping cañon sides of the San Diego River, and requiring a number of trestles and tunnels in its course in order to cross side-drainage channels, or to pierce long bends which would have greatly increased its length. In all there are 315 trestles, having a total length of 38 042 feet and heights ranging from a few feet to 85 feet, the longest being that across Los

Coches Creek 1 794 feet in length (Plate XXXIX) shows the work during construction. There are also eight tunnels pierced in rock or earth with lengths varying from 9 to 1 901 feet, the total length of which is 4 190 feet, with 4 760 additional feet of masonry approaches.

The Cuyamaca Reservoir, situated on the head-waters of Bowlder Creek, a branch of the San Diego River, has a catchment basin of 11 square miles on which the annual average precipitation is about 45 inches. This reservoir is formed by a dam 700 feet long and 35 feet wide on the crest; 140 feet wide at its base and 35 feet in maximum height. The water slope is 1 on 2 and the outer slope 1 on 1½, while the inner slope is covered with a rip-rapping of stone 2 feet in thickness. This dam is composed of a hard clay which was accidentally found below the surface while excavation was in progress for the foundation. It rests on this foundation bed of clay which is over 12 feet in depth, into which a trench 8 feet in depth is excavated and filled with a puddle of one part of small broken stone, one part sand and three parts clay, the whole well rammed. This puddle wall is carried up through the entire height of the dam, having a top width of 6 feet and a batter on both faces of 4 on 1. The surface of the ground under the inner slope of the dam is covered with 2 feet in depth of clay puddle well bonded. The chief element of weakness in this dam is the discharge sluice, consisting of a trench filled with puddle, on which 18 inches of concrete is laid, and above which the masonry culvert is built up. Owing, however, to bad bonding with the surrounding material of the dam, a considerable amount of leakage occurred at first, following the line of this culvert. This has been recently diminished, however, by the addition of ribs to the outer circumference of the culvert and the adoption of other remedial measures. A little beyond the south end of the dam a waste-way is constructed 50 feet in width and 4 feet deep, through which the velocity of the water is about 5 feet per second. This waste-way is ample to discharge the greatest flood which may fall upon the catchment basin of the reservoir.

After flowing down Bowlder Creek and the San Diego River for 12½ miles the water is diverted into the San Diego Flume by means of a great pick-up weir built of uncoursed rubble masonry, which extends across the entire width of the San Diego River. This weir shown in Plate XL is built in two tangents, the exterior angle of which points up-stream. Near its south end and extending out



ERECTION OF HIGH TRESTLE ON SAN DIEGO FLUME, CALIFORNIA.

PLATE XL
TRANS. AM. SOC. C. E.
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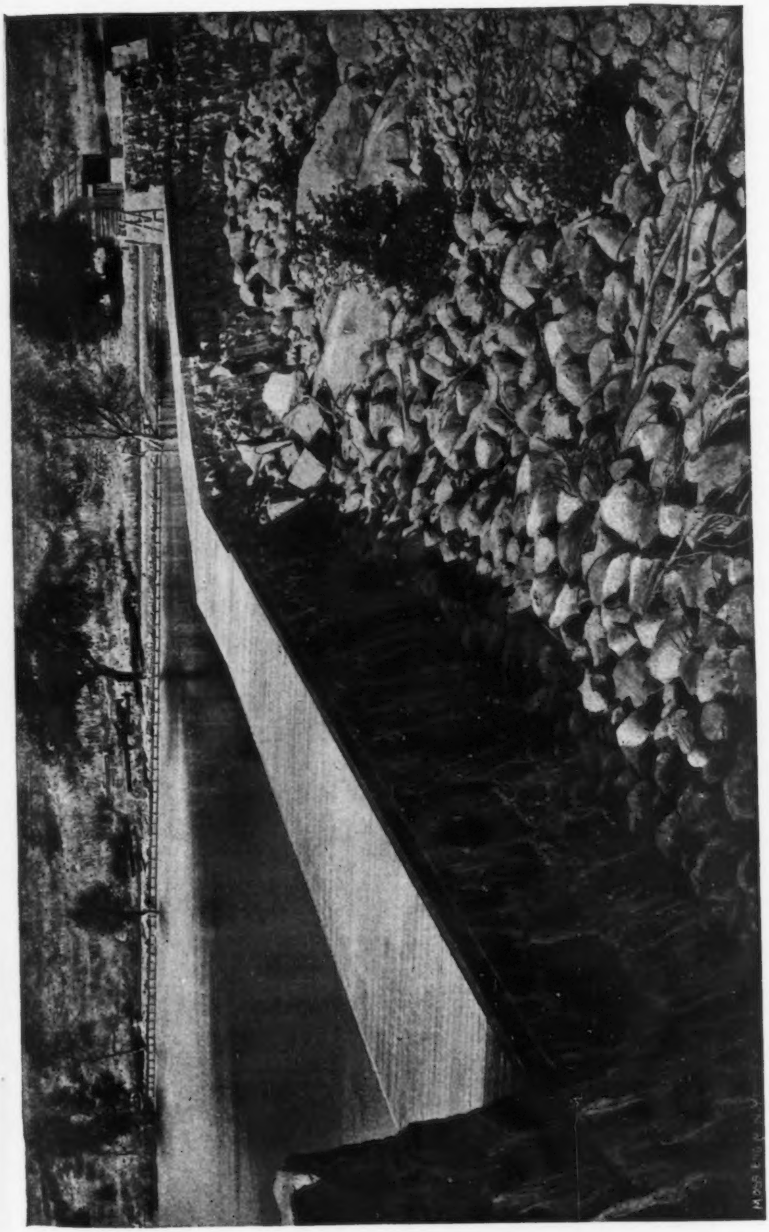
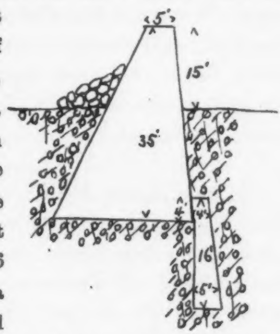
WILSON ON AMERICAN IRRIGATION.



DIVERSION WEIR, SAN DIEGO FLUME.

for a distance of 108 feet to the head or regulating gates which admit the water to the flume, the cross-section of this weir is 4 feet wide on top, or 1 foot less than that of the remainder of the weir. At a distance of 32 feet beyond the flume head is an open waste-way 20 feet wide, the crest of which is 4 feet lower than that of the remainder of the weir. For 14 feet beyond this waste-way the weir is again given its usual height as far as another waste-way 165 feet in length, the crest of which is, like that of the first described, 4 feet lower than that of the remainder of the weir; the remaining 15 feet in length of the weir is again full height. Between the two waste-ways just described and in the bottom of that portion of the weir which is built to the maximum height, also, under the regulating gates at the head of the flume, are two single under-sluices opened by means of gates which slide vertically and are operated by means of a screw and hand-lever from above. One of these under-sluices has its sill 14 feet and the other 18 feet below the crest of the dam. The cross-section of the weir, a diagramatic sketch of which is shown, is peculiar, and is the result to a certain extent of a failure on the part of the builders to found it on bed-rock. The first wall as originally built was 35 feet in height, 5 feet wide on the crest and 16 feet wide on the base, and was sunk to a depth of from 15 to 25 feet into the gravel of the river bed, presumably to bed-rock. It was soon discovered, however, that a considerable leakage passed under this weir, and after examination by competent engineers it was decided to build a sub-wall on the up-stream side across the deepest part of the river channel as indicated. This wall is 16 feet in height, but it is claimed by those who were present during its construction that it was never carried down all the way to bed-rock. This belief is further exemplified by the fact that there appears to be considerable leakage under the weir, as the discharge of the river below it is claimed to be greater than it was before the weir was constructed.



The regulator at the head of the flume consists of two gates, the sills of which are about 10 feet below the crest of the weir. These are 40 inches each in the clear, and are raised by means of a screw and hand

lever operated from above. The flume which transports the water to the irrigable lands in the neighborhood of San Diego is well constructed, is entirely of wood, and is everywhere laid in excavation or on trestles, in no place resting on embankments. The excavation in which it is laid is 12 feet wide, or 6 feet wider than the flume, thus giving a place in which loose rocks and stones sliding from the hilltops above can lodge without injury to the structure. The flume is 6 feet wide in the clear with sides now 16 inches high, though the side posts and framing are 4 feet in height, so that whenever the demand for the water requires it, two further rows of planking will increase the capacity of the flume by that additional depth. The flume rests on transverse sills of 2-inch planks laid 4 feet apart, and on these rest 4 x 6 longitudinal stringers, above which is constructed the frame work of the flume with its lining of 2-inch planking. At a point about 200 yards below the head-gate is constructed an escape or waste-gate, 4 feet in width, which discharges into a short flume 50 feet in length, by which the waste water is led back to the San Diego River. The grade of the flume is 4.75 feet per mile, its maximum capacity is estimated to be 50 second-feet, and its total length is 34.45 miles.

Another and the most modern storage reservoir constructed in the west, the dam of which is composed of earth, was built by the Crocker-Huffman Land and Water Company, about 5 miles northeast of Merced, Cal. This project consists of a temporary diversion weir on the Merced River, about 25 miles above the city of Merced; of a canal 27 miles in length, which leads the water into the reservoir; of the storage reservoir itself, and of the distributing canals leading from it to the irrigable lands. The Merced reservoir, the surface area of which is 500 acres, is closed by a dam 4 000 feet in length, and 54 feet in maximum height, the top width of which is 20 feet, and the capacity 15 000 acre-feet. For 2 600 feet of its length the dam is quite high, the remaining 1 400 feet being a mere embankment not over 10 feet in height. Its inside slope is 3 to 1 and the outside slope 2 to 1, while the inner slope is rip-rapped for 12 inches in depth with stones roughly laid on for a depth of 15 feet below the crest. The catchment area of the reservoir proper is so small that it was not considered necessary to provide any other escape than the outlet tunnel of the reservoir, which has a capacity of about 100 second-feet. A mile above the reservoir on the main canal line is an escape discharg-

ing into a natural channel, and at an elevation of 1 foot above the canal sill at the point where it discharges into the lake, so that if the lake were exceptionally high, this escape would be brought into operation. According to Mr. C. D. Martin, the engineer who constructed this work, the dam is composed of a sandy clay laid dry in layers, and well tramped over with scrapers and horses. The foundation is of the same material, and test borings showed its depth to be at least 15 feet. The discharge pipe is 2 feet in diameter, and is constructed of ½-inch cast-iron laid in a brick-in-cement tunnel 4 feet in diameter, with walls 1 foot in thickness. Every 15 feet on the outer circumference of the tunnel are projecting rings of masonry 1 foot in thickness and 2 feet in height, the object of which is to prevent the travel of seepage water. In addition to the outlet pipe which carries water under pressure to the city and to some of the surrounding irrigable properties, there is at the south end of the dam an ordinary discharge gate, set a few feet below the maximum surface level of the reservoir, which leads into a distributing canal commanding a large portion of the irrigable lands.

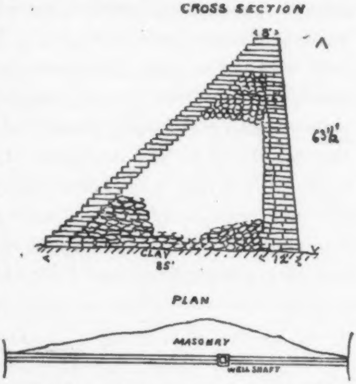
The headworks of the feeder canal are of the simplest and crudest description, consisting of an old weir built of brush, rocks and crib-work, the total length of which is 250 feet, and the height 12 feet. The canal heads 200 feet above the dam at a slight angle pointing upstream, and its entrance is closed by a set of sixteen regulating gates, each 5 feet wide in the clear and 20 feet in height above the canal bed. This regulator, which is formed somewhat like a long flume with gates in it, has a floor of 3-inch planking extending 20 feet upstream and 10 feet down-stream. Anchor and sheet piling is let 4 feet into the rock and cemented under the upper and lower ends of the floor and under the gates, the latter being simple wooden structures which are lifted by means of a hand-lever operated from above. The first main portion of the canal, which is 7 miles in length, has a capacity of 1 500 second-feet and a bed width of 60 feet, with slopes of 2 to 1 inside, and 1½ to 1 outside, a depth of water of 8 feet and a grade of 1 foot per mile. This portion of the canal is excavated mostly in sand and gravel, while the remainder of the canal is generally in a fairly good soil. On the line of the canal are two tunnels, the first 1 700 and the second 2 000 feet in length, both are 20 feet wide at the bottom, 14 feet in height at the summit of the arch, with a grade of 10½ feet per mile. The first tunnel is excavated entirely in sandstone and requires no lining, while

the second is in friable sandstone or in gravel, and is timbered throughout. Below the first tunnel is taken off the first main distributary which discharges into a creek, the line of which is utilized as a canal. The practice of utilizing stream beds as distributary channels is employed generally on this canal, and is not one that is to be commended. Experience has shown both in this country and abroad, that the loss by evaporation and absorption is greater in a natural channel, the grade and bed width of which is not exactly that which theory would require, while the difficulty and expense of diverting water from such stream beds to the private irrigating channels form an additional objection to their use. Moreover, the employment of the natural drainage lines of the country as irrigating canals impedes the proper circulation of the irrigation waters and gives rise to the production of alkali and malaria.

A very interesting composite irrigation scheme, combining, as it does, two different varieties of water storage, is the project of the Denver Water Storage Company, the main reservoir for which has just been completed. This company acts in co-operation with two other corporations, one of which, the Denver-Arapahoe Land Company, controls the lands which this project will irrigate. The system comprises a main storage reservoir, known as Castlewood Reservoir, situated on Cherry Creek, at a narrow point in the cañon and about 30 miles southeast of the City of Denver; of a pick-up weir a mile and a half lower down on Cherry Creek which will divert the water which is turned into Cherry Creek, and pass it into the Arapahoe Canal which heads at that point; of the main line of the Arapahoe Canal which conveys the water to the irrigable tract situated immediately southeast of Denver and West of Cherry Creek; and of a series of four secondary reservoirs situated in different parts of the irrigable lands which are formed by natural depressions, and one of which has been completed and is now being utilized. This system of secondary reservoirs will be filled by water brought down by the canal at such times as it is not wanted for the irrigation of the land, or when there may be a superabundant flow in Cherry Creek which would otherwise be lost. The area of good irrigable land which is commanded by this system as at present constructed is 30 000 acres, and the approximate cost of the canal and main reservoir is about $305 000.

The capacity of the reservoir is 5 300 acre-feet, and its catchment area is 200 square miles, the available run-off of which is estimated to be

about 2 300 acre-feet per annum, while the maximum flood anticipated is 7 000 second-feet. The construction of the dam is peculiar (see sketch) and has been severely criticised by the people living below it and by the inhabitants of Denver as unsafe, and injunctions have been taken out to prevent its being filled with water. It is founded on a bed of clay and bowlders from 7 to 30 feet in depth, and is composed of an outer shell of large coursed blocks of rubble masonry, the thickness of which on the up-stream face is about 6 feet on top and 12 feet at the bottom. On the down-stream face it is from 5 to 7 feet in thickness, this facing being laid in steps as shown in the accompanying diagram, the height of the steps varying from 1½ to 2½ feet. The main body or center of the dam consists of dry-laid rubble and stone, inclosed between these two masonry shells. The maximum height of the dam is 63½ feet, its crest is 586 feet long, and 100 feet of this length is lowered 4 feet in order to form a waste-way over which flood waters may be discharged. In addition to this waste-way is another around one end of the dam, which is 10 feet wide and has a fall of 4 feet in its length of 400 feet discharging back into Cherry Creek below the dam. The upper 4 feet of the dam is vertical on both sides, 8 feet in width, and constructed of rubble masonry, the outer slope of the remainder of the weir is 1 on 1, while its inner slopes is 10 on 1. The well-shaft for controlling the outlet sluice is constructed of the best masonry and is built into the dam close to its upper slope, the total thickness of the masonry wall on both sides of this shaft being 4 feet. Discharging into the shaft are four sets of iron intake pipes set in cement, the lower set being on a level with the bottom of the reservoir and the others each 6 feet apart vertically. These discharge into a central well from which an outlet pipe 3 feet in diameter leads the water off, the pipe being surrounded by 4½ feet in thickness of concrete.



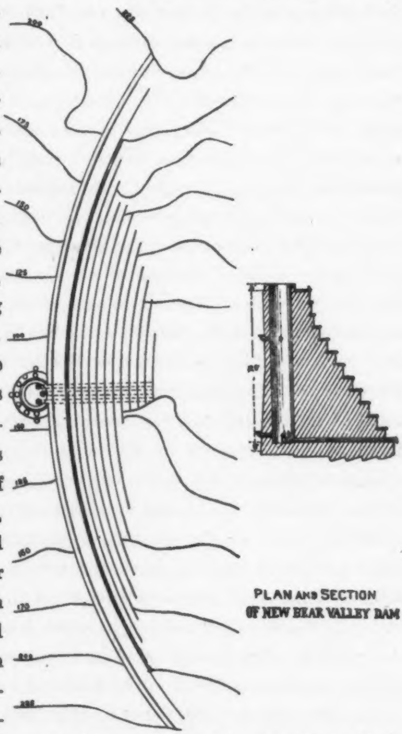
After being discharged into Cherry Creek the stored water runs down

the bed of that stream for a distance of 1½ miles to the pick-up weir, which diverts it from the left bank into the canal. This weir is 150 feet long, 10 feet high, and, with the exception of 54 feet of its length in which the head-gates are situated, it is composed of piling sheeted on both faces and forming a rectangular wall which is filled with sand. These piles are 26 feet long, rest on bed-rock, and are placed 6 feet apart across-stream, and 3 feet between centers in the direction of the stream. Through the center of this portion of the weir is a waste-way 60 feet in width, with its crest 3 feet lower than that of the remainder of the weir. The head-gates, as before stated, are taken out of that portion of the weir which is constructed of rubble masonry, and are three in number, their total height being 6 feet, and their width between centers 4 feet. The masonry section of the weir in which these gates are located is 3 feet wide on top, its inner slope being 1½ on 1, and the outer slope vertical.

The canal by which the water is led into the secondary reservoirs and to the irrigable lands, has a total length of about 50 miles including its main branches; the first 25 miles of this line is 12 feet wide on the bed, has 3 feet depth of water, slopes of 1 on 1½ and a grade of 1½ feet per mile, giving it a discharging capacity of 75 second-feet. In the next 10 miles the canal has a bed width of 10 feet and the remainder of its course has a bed width of 8 feet, the slopes and grade remaining the same. The majority of the line of this canal is constructed in an easily worked clayey loam.

The secondary reservoir which is already constructed has a surface area of 60 acres and a capacity of 700 acre-feet, and as before stated, consists of a natural depression the maximum depth of which is 16 feet. This reservoir is emptied by means of a wooden outlet pipe let into the embankment which closes the lower side of the basin. This earthen embankment is 10 feet in maximum height, 12 feet wide on top and has slopes on the inside of 1 on 3 and outside of 1 on 1½. On the line of the feeder canal are twenty flumes crossing smaller side drainage lines, the greatest being 200 feet in length and the greatest height 12 feet. Nearly all of these flumes are furnished with escape gates by which surplus water can be discharged into the creek. In addition to these there are 5 300 feet of inverted wooden syphon pipe line for crossing ditches or drainage, the longest syphon being 1 800 feet in length with a maximum depth of 100 feet, giving a water pressure of 50 pounds per square inch.

The Bear Valley Dam, which is situated in the San Bernardino Mountains east of San Bernardino, Cal., has been so frequently described because of the peculiarly bold cross-section given to it, that it will be unnecessary to refer here to any of the details of its construction. In connection, however, with the company which built this work, there have recently been inaugurated a series of large irrigation projects which will necessitate the construction of a greater storage reservoir than that now existing. The company which is now developing this property is known as the Bear Valley and Alessandro Development Company and is under the general management of Mr. Frank E. Brown, the builder of the original Bear Valley Dam, who also acts as consulting engineer for the new work. This company owns a large tract of land about 10 miles south of the City of Redlands, from which it is separated by a low range of hills. The water supply, however, must be derived from the Santa Anna River and its tributaries which flow through Redlands and on the headwaters of which is located the storage reservoir. The perennial discharge of this river is comparatively small, and is already entirely appropriated by companies having prior rights and operating in Redlands, Riverside and other localities; it accordingly became necessary to obtain storage water for the irrigation of the Alessandro tract.



PLAN AND SECTION OF NEW BEAR VALLEY DAM

The capacity of the present Bear Valley Reservoir is said to be 36 000 acre-feet. It is situated in a broad and flat mountain basin known

as Bear Valley, at an altitude of about 6 200 feet above the sea, and discharges into Bear Creek, a fork of Santa Anna River, through a very narrow granite gorge in which the dam is built. This dam (see sketch), which is founded on granite and abuts against the firm granite walls of the cañon, is 300 feet in length on the crest, and is curved with the convex side up-stream in the form of an arch, the radius of which is 335 feet. The cross-section of this dam is remarkable; its maximum height is 64 feet, the top width 3 feet and the lower face vertical for 48 feet, the upper face being battered so that at a depth of 48 feet it is 8.5 feet in thickness. At this plane there is an offset up and down stream giving the crest of this lower wall a thickness of 12½ feet. Below this plane both faces have a slight batter to the foundation, which has a thickness of 20 feet. The structure is constructed of rough ashlar masonry on both faces, filled with coursed rubble masonry in the interior, all laid in uniform beds of Portland cement. These stones are of granite and vary from 2 to 5 feet in length and about 1 to 2 feet in width.

The south end of the dam abuts against a massive ledge of granite standing out about 100 feet into the cañon. This ledge really forms a part of the dam and over it an escape-way has been cut having a width of 20 feet, with its sill 8.5 feet below the level of the crest of the dam. Through the bed rock, about 9.5 feet below the base of the dam, and one-third of its length from the southern end, is a cutting which forms a masonry-lined culvert 2 x 3 feet in dimensions which discharges into a masonry pool from which it is expected that the water can be measured over a weir. On the upper side this culvert narrows to its entrance which is closed by an iron gate 10 x 24 inches, which slides on brass bearings and is operated from above by means of a screw and wheel. Frail as is the cross-section of this structure, it is a remarkable fact that owing to the excellent manner in which it was built, and to the perfection with which it acts as an arch, it has stood now for several years, through as many years of heavy floods and shows no sign of weakness. The waste-way is ample for the discharge of all the flood waters, though enormous floods occasionally occur at the high altitude at which it is located. During the twenty-four hours included from 6 P.M. on February the 26th, to 6 P.M. on February 27th, 1891, a rainfall of 17 inches was recorded on the rain-gauge at the dam, while the total downfall which occurred during that week was, perhaps, as much as 40 inches.

The new dam, the money for the construction of which has already

been subscribed, will be a much larger and more substantial structure than the present one, and will be situated 150 feet below it in what is considered a more favorable location. Like the present dam it is to be constructed of the best granite masonry, and will be arched with its convex side pointing up stream. This dam is to be 120 feet in maximum height, the foundation being from 8 to 12 feet deeper and the coping 5 feet in height on the top of the crest. Its upper surface will have a very slight batter, and the lower surface, while its general slope is about the same as that which would be given by Molesworth's, Bouvier's, or some other of the modern formulas, will be built in steps having a uniform rise of 10 feet and an offset in proportion to the batter at that point. The top width of the dam will be 15 feet, its width at base 73½ feet, and the maximum pressure when full is estimated to be 11.6 tons per square foot on the toe of the dam, and when empty 9.3 tons per square foot at the upper side. The discharge sluices, which are three in number, head in a cylindrical gate-well which extends the entire height of the dam on its upper face. This gate-well will be 18 feet in diameter inside, and will have three inlet pipes at the bottom, four more 40 feet higher, and three more at a height of 80 feet from the bottom. The outlet sluices will consist of three cast-iron pipes, each 36 inches in diameter, the entrances to which will be closed by gates worked from above, and the total discharge of which is estimated to be 1 500 second-feet.

This great dam will flood the water back from Bear Valley proper over a low divide, into a smaller basin which is situated on the eastern watershed of the San Bernardino Mountains. The area which will be flooded will be 7 850 acres, and the maximum capacity of the reservoir is estimated to be 322 000 acre-feet. The waste-weir for the discharge of surplus water in time of flood will be located at the opposite end of the valley from the dam at the lower end of the second basin, and will discharge toward the eastern plains. The capacity of this reservoir is so great that the watershed of Bear River, which is only 77 square miles, is insufficient to fill it; accordingly a canal will be constructed along the eastern slope of the mountains which shall catch the water of White River, Mission Creek and several minor streams, and lead it into the reservoir at its eastern end. This feeder canal will have an ample cross-section to carry the ordinary flood water of these streams and will be about 18 miles in length.

When wanted for purposes of irrigation the water stored in this re-

servoir will be discharged into Bear Creek, down which it will flow a distance of 12 miles to the Santa Anna River, and an additional 15 miles down this stream to its exit from the foot-hills, at which point it will be diverted. In its course down these streams the water flows over a very steep bowlder-bed with a total fall of about 5 200 feet. At present there are diverted from the Santa Anna River two small canals, one from either bank, which lead the water to the irrigable lands in the neighborhood of Redlands. The new project will carry the water, by means of a pipe-line 10 miles long, to the Alessandro tract.

This pipe-line is now in process of construction and is nearly completed. The diverting dam at the head of the pipe-line has not yet been constructed but it will be a substantial and well designed structure, and immediately above it the pipe-line heads, having a perfectly straight line for a distance of about 2¼ miles to the crossing of Mill Creek, in which distance it passes over some hilly country. From there on it follows somewhat the contour of the country to the divide before spoken of as separating the Santa Anna Valley from the Alessandro tract. This pipe-line is of steel, 28 inches in diameter at first and afterwards reduced to 24 inches. The tunnel through the divide is 2 300 feet in length. It is cut through hard rock, the floor and sides of which will be lined with a coating of asphaltum. Beyond the tunnel will be 2 600 feet of open canal, likewise asphaltum lined, after which will be laid about 150 miles of distributing pipe, some 14 inches in diameter, and the remaining branches about 9 inches in diameter. It is intended to pipe this water to the highest point of every 10-acre lot, whence it will be distributed to the plants by means of sub-irrigation through pipes, as is done in many other parts of Southern California.

It is not necessary in this article to more than revert to the Sweetwater Reservoir, which is situated a few miles southeast of San Diego, California, on the Sweetwater River. The magnificent dam by which this reservoir is formed has been ably described by Mr. James D. Schuyler, the engineer who constructed it, in a paper read before this Society in November, 1888.

The latest and most modern reservoir closed by a masonry dam which has been designed in this country is that on the Sharon estate in California, known as the Berenda-Chowchilla project, of which Mr. William

Ham. Hall is chief engineer. This tract consists of 30 000 acres for which a storage supply must be provided, as the perennial flow of the Chowchilla River is not sufficient to irrigate it. The project consists of a storage reservoir situated on Chowchilla Creek, from which the water will be discharged into that stream, and from which it will be diverted after flowing down it for 2¼ miles by a diversion weir; and of the main diverting canal which will lead the water to the irrigable lands. Though the perennial discharge of the Chowchilla River is small it discharges in times of flood as much as 1 500 second-feet, an ample volume to fill the reservoir within a few weeks.

The dam which is to be constructed of uncoursed rubble masonry is at an altitude of 420 feet above the sea and will form a reservoir having a superficial area of 100 acres and a capacity of 42 400 acre-feet, which it is believed will be sufficient to irrigate 26 000 acres of the tract. The maximum height of the dam is 100 feet, its length on crest is 780 feet, and it is curved up-stream with a maximum radius at the center of 1 146 feet, which radius is diminished gradually to 736 feet at the abutments, thus broadening the base on which it abuts against the hillsides, and thus increasing its power to act as an arch. Two waste-ways will be excavated, one at either end of the dam, the combined discharge capacity of which will be 15 000 second feet. The entire structure is founded on a firm dioritic rock which outcrops near the surface, and while its cross-section is very similar to that given by Wegmann's or other modern formula, it is made a little heavier than these would require, owing particularly to the increased width near the abutments. It has a uniform batter on the up-stream face of 1 in 25 below the first 10 feet, which are vertical, its down-stream face being curved according to theory and producing a maximum width at its base of 68 feet. The top width of the dam is 10 feet, which will be widened by a cornice and a 4-foot parapet to 13½ feet. There will be two outlets or discharge channels excavated in solid rock under the dam at either side of the stream-bed, and each 20 feet above its base. These discharge sluices will each contain a steel pipe laid in cement and operated by valves from above. In addition to these there will be an under-sluice 6 feet above the bottom of the dam, also laid 10 feet deep in the rock under the structure and in which will likewise be laid a steel pipe.

Water will be diverted from Chowchilla Creek by means of an uncoursed rubble masonry weir 12 feet in maximum height and 400

feet long on the crest; this will be a clear overfall weir, and is located on a reef of rock in which is a natural depression at the south end of the weir, which will be utilized as the canal-head. This depression is 30 feet in width below the crest of the weir and the main canal will head at this point. This canal will have a capacity of 300 second-feet and will be 4½ miles in length with no difficulties of construction on its line. At the end of this distance it will be divided into several distributing branches which will convey the water to the irrigable lands.

In addition to the works described in this article there are numerous other important projects either completed or under construction which illustrate the magnitude and importance of the present stage of irrigation developed in our West. Some of these works are quite as large and as well worthy of note as those which have been described, though few of them contain as many important and typical engineering works as those mentioned. Among the more important canals may be mentioned the Eddy Canal, which is diverted from the Pecos River in New Mexico, at the head of which is a great diverting dam constructed of loose rock with an earth backing, the maximum height of which is 54 feet and the length on the crest of which is 1 100 feet. This dam also creates a storage reservoir which supplements the discharge of the river. The canal itself is 50 feet wide and 6 feet deep, about 30 miles long, and on its line are some very heavy rock works and a high terre-plein and aqueduct crossing the Pecos River.

Another important canal, which is also nearly completed, is diverted from the Sacramento River above the Town of Colusa in California, for the irrigation of the Central District. This canal has a capacity of 750 second-feet; is 60 feet wide at the bottom; carries 6 feet in depth of water, and will command about 156 000 acres. The first 6 miles of its length are in earth excavation, the maximum depth of which is 19 feet, and at its head and on its line are several important and substantial structures, such as regulating gates, falls, etc.

Among the other important reservoir projects of which no mention has been made, is that of the Citizens' Water Company, near Denver, Colo., the primary object of which is for the water supply of the City of Denver. This reservoir will be closed by a mammoth earth dam 261 feet in height and 30 feet in width on top; the side slopes of which are 2½ to 1 inside and 1½ to 1 outside, set back with several benches or bermes, each 10 feet wide, to prevent slips.

DISCUSSION.

Colonel William E. Merrill, M. Am. Soc. C. E.—Is there any difference in the effect of irrigating with clear or with muddy water?

Mr. Wilson.—There is a considerable difference of opinion on that subject. The Indian practice is to try to get the clearest water they can. They use their main canals very much as reservoirs and take their drainage canals off at some feet above the level of the main canal, as high as they can, in order to get the clearest water possible. In our Western States the practice is exactly the reverse of this; they do not object to having sediment in the canals; they think it helps the canals, providing they can keep the sediment in suspension, and allow it to be deposited on the lands. In the case of the Turlock canals, the main canal, its branches and ordinary distributaries are so arranged, that the sediment will be held in suspension until it reaches the irrigable lands, as this is considered desirable.

A Member.—What is the usual method of utilizing water from the canals? Do they overflow the land with the water, or simply dig a few ditches and let it go through the soil?

Mr. Wilson.—I have not touched upon that point because I considered it was not connected with irrigation engineering. As the article is rather long, I have avoided touching upon a good many features of irrigation.

The practice differs considerably in different parts of our country, but the water is utilized, I may say, by three methods. In a gently sloping country, like California, the check system is most employed. This consists of building up a series of embankments about 1 or 2 feet in height, by means of scrapers and plows. The water is poured from the ditches above each successive check, by which it is held up and is allowed to soak into the soil. In most parts of the country the general practice is to construct little branch canals to the various plots to be irrigated, and if the crop is wheat or grain or grass, they so plow their fields as to leave ditches about 10 or 12 feet apart, into which water is turned. These are laid out on a proper grade. The water goes into them and is absorbed by the soil; then they break away the next check, and allow it to flow down to the next ditch. The second method is to flow the water over the land and to a depth of 6 inches, and allow it to lie and to soak into the ground. The practice is not to irrigate more than three to five times a season.

In Southern California subirrigation is now rather extensively practiced, by a system of pipe lines that lead to every individual tree and have an outlet under the tree, and the water is absorbed in the soil by capillary attraction; in that way the duty for water is the highest

that is obtained anywhere. In the case of the rice crops in India the land is flooded to several feet in depth.

JAMES B. FRANCIS, Past-President Am. Soc. C. E.—What means do they take in the earthen dams to secure tightness? By means of core walls?

Mr. WILSON.—The practice in the western States is similar to that of Europe and India. I know of no earth dam in the west which has either a masonry or a wooden core. I do not know of any work which is considered a good work which has a puddle wall in it. Sometimes they excavate puddle trenches, down to some impervious strata of clay, and the puddle wall is carried up a few feet into the embankment, but the embankment itself is usually built in layers and consists of a clayey soil, wherever possible, with gravel and small stones mixed in. In the case of the Cuyamaca dam there is a certain proportion of clay—apparently mixed with gravel—and small stones, properly mixed. In the case of the Modesta dam, where floods are not feared, the common soil was tramped over and worked down in layers.

A MEMBER.—I understand they do not roll the layers.

Mr. WILSON.—They did not.

WILLIAM P. HARRIS, M. Am. Soc. C. E.—I understand that the slopes are 1½ on one side and 2½ on the other.

Mr. WILSON.—Not on the reservoirs; on the canals the slopes are steep. On the reservoirs mostly 2½ and 3.

Mr. HARRIS.—That seems to me astonishing. The levees in the Mississippi River are built entirely now of slopes of 1 on 3, and engineers prefer to build them 1 on 4. They are pretty well trodden down and tamped, but they are given a slope of 1 on 3, sometimes 1 on 4 and even 6; in case of extreme height they have a bank on the land side of about 20 feet, but no such slope as 1 on 1½. The material varies from a pretty clear sand to a pretty clear clay.

G. BOUSCAREN, M. Am. Soc. C. E.—What kind of timber do they use in a flume canal and how long does it last?

Mr. WILSON.—It differs considerably with the locality. In California, take the San Diego Flume Company's work—the trestle and the sills and all the timbers on that work are of Oregon pine, and the planking is of redwood. Redwood is supposed to last for a very long time. The San Diego flume has been constructed about eight years and it does not show the slightest sign of deterioration so far as I was able to discover. It is laid everywhere in excavation on a narrow embankment, and there is about 4 feet width of excavated berme on which it rests on each side of the flume. The bench flume in the Highline Canal in Colorado has been in operation about ten years. It is built, I think, of Colorado pine, and that flume shows no great deterioration of the trestles and other timbers. A good deal of the planking has been replaced, but I suppose

a very small percentage of the entire flume has been repaired. I should judge their life would average ten to fifteen years.

John C. Trautwine, Jr., Assoc. Am. Soc. C. E.—Mr. Wilson mentioned a great difference in the expansion and contraction of the aqueduct; how was that?

Mr. Wilson.—They have overcome it, as I recollect—(I do not wish to be put down as saying it is so)—largely by building the wooden flume well into the iron aqueduct and by making leaden connections. The engineer in charge criticised the iron aqueduct, and said it ought to be replaced by a wooden one. The great aqueduct is simply an iron trestle on which rests a wooden flume, so that the water channel is a wooden flume throughout, both on land and on trestle.

Colonel Merrill.—Judging from the experience of others I should conclude that there was no insuperable difficulty in the use of iron aqueducts. I have in my office a photograph of an iron aqueduct on a navigable feeder of the Central Canal of France, and from what I know of the minute attention given to details by French engineers, I should conclude that satisfactory means had been found for making the ends water-tight. The clear width is 10 feet and the depth of water is 5 feet. At the famous hydraulic lift at Fontinettes (also in France) the upper approach consists of two independent metallic aqueducts, with a span of 68 feet, a depth of water 6½ feet, and a clear width of a little over 17 feet. When I visited this work in 1889, I neither saw nor heard of any leak. In our own country there is an aqueduct on the Mussel Shoals Canal, where it crosses Shoal Creek, that is 900 feet long, 60 feet wide in the clear, and carries 6½ feet of water. I have never heard of any difficulty in keeping the ends water-tight.

Mr. Wilson.—On the Agra Canal in India, is an iron aqueduct, 10 feet in height and 20 feet in width, which is connected with the river and masonry abutments by means of sheet lead. There is no leakage of any consequence on this aqueduct.

Mr. Francis.—I have had occasion to make large lines of pipe, from 3 to 6 feet in diameter, of boiler iron; these are connected with masonry at one end. Expansion would be a great detriment, so we provided for it by riveting to each of the pipe ends to be joined a plate with a wide flange turned outward at right angles. These flanges were riveted together near their edges, thus allowing for a certain amount of change of length.

Mr. Trautwine.—Is not that the same form of joint that is used in the larger water pipes in Calais, France?

Mr. Francis.—I do not know, sir, what they use.

Edward P. North, Director Am. Soc. C. E.—There is one question I would like to ask about. Mr. Wilson speaks of a fill on the Bear River Canal 108 feet in height. There should be a pretty large allowance for

shrinkage there. What effort have they made to allow for shrinkage and how successful was that effort ?

Mr. Wilson.—There are several such fills on the line of the Bear River Canal. These fills are across drainage lines and the water which might flow down the drainage lines was carried through the fill in a culvert. Of course the deep fills would settle considerably. They were all allowed to rest about a year before the wooden flume which carried the water was laid on top of them. At the same time, subsidence will doubtless take place, and several of these fills have been seriously damaged by leakage from the flume. I think it is perhaps unusual to put a wooden flume on top of such a fill, the width of which hardly exceeds the width of the flume. I saw two of these flumes in February, when, after the water had been turned into the canal, the fill had been washed away and flumes and fills had all gone out. Piles were driven into these fills on which the flume rested, but the piles had not been driven to the solid ground surface and were not of much service. They are now caulking the flumes and hope to keep them in shape in that way.

Robert Moore, M. Am. Soc. C. E.—I don't know anything that better illustrates the need of good engineering and the damage done by bad engineering than some of the works that have just been described. One of them happens to have come within my notice, namely, the Arizona Canal. Some years ago I visited the head works of this canal, and found it to be an admirably conducted work, with the single exception of the diversion dam. This had been recently swept away and the whole work thereby rendered useless. The second dam constructed since then is certainly a mistake; both the place and the manner of its construction being faulty. The dam mentioned as the third is in the proper place and is the only one that should ever have been constructed. The whole work in this respect has been a series of mistakes. It is very strange that people will invest so much money and risk it upon such flimsy constructions as these. The western country is now infested with ditches that carry no water. If something can be done to correct this evil it will be of great value to the country.

Mr. Wilson.—I would like to say in defence of the engineer who designed the head works of the canal, when he found that I was going to look over them carefully, he told me the form of construction was one that he protested against, and had informed the company that it would be carried away, but the pressure brought to bear on him was such that he had to build it in a very short time and in that place. He would have placed it where it is now to be placed, where the head works of the canal would be good, and where the dam itself would have good abutments and foundation, but he could not build it there inside of eight months, and they told him it had to be built right away, so he built it right away, though he was well aware of its instability.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

493.

(Vol. XXIV.—August, 1891.)

SOME EXPERIMENTS ON THE TRANSVERSE BREAKING STRAIN OF PLATE GLASS.

By G. W. PLYMPTON, M. Am. Soc. C. E.

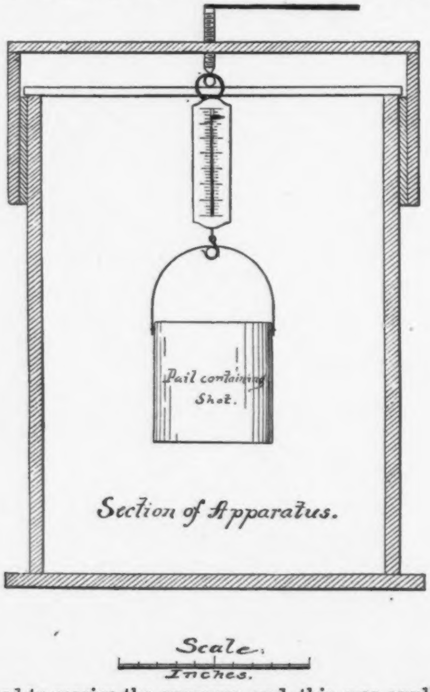
WITH DISCUSSION.

Having occasion some three months ago to determine the pressure which plate glass windows would safely withstand when exposed to strong winds, I consulted numerous tables to ascertain what had been recorded as determined transverse strength. Only two or three such records were found and these were discordant; naturally so from the nature of the material. I resolved therefore that I would subject specimens of the glass to be used in the structure to the test.

The specimens tried came from Holland. They were of a uniform length of 20 inches, and a width of 2 inches, and the thickness varied as shown in the table. Every strip was broken by weights added at the center from a relatively small weight up to that producing rupture. The strip or bar of glass rested on upright supports just 18 inches clear. A bar of half-inch round iron was placed across the middle of the test piece and projected far enough on each side to permit a spring balance to be hung on each end. A second bar of the same size was placed in the lower ring of the balances, and this bar supported a pail into which shot was poured for the trial weights.

To measure the deflections a wooden cross-bar was placed above the glass strip. Through the center of this bar a pointed screw of thirty-

two threads to the inch projected, and was of sufficient length to follow the deflections. This screw was run down at each measurement so as just to touch the bar on the middle of the glass strip. To insure regularity in the pressure of the screw upon the rod a strip of paper



Section of Apparatus.

was interposed to receive the pressure, and this was applied so as to permit the paper to be gently pulled between the screw point and the bar. The deflections were taken by counting the turns of the screw, the fractions being made apparent by a long horizontal indicator wire soldered to the head of the screw.

	Thickness, inch.	Weight, lbs.	Deflection, inch.
Experiment No. 1............	.315	16	.035
....		20	.043
....		24	.053
....		28	.059
....		32	.066
....		36	.074

	Thickness, inch.	Weight, lbs.	Deflection, inch.
Experiment No. 1............		40	.082
....		44	.090
....		46	Broke
Experiment No. 2............	.315	16	.037
....		20	.045
....		24	.051
....		28	.059
....		32	.066
....		36	.074
....		40	.082
....	.315	44	.090
....		48	.098
....		49	Broke
Experiment No. 3............	.345	16	.027
....		24	.049
....		32	.058
....		40	.062
....		47	Broke
Experiment No. 4............	.335	20	.043
....		30	.059

Left under this stress broke in four hours twenty minutes.

	Thickness, inch.	Weight, lbs.	Deflection, inch.
Experiment No. 5............	.295	8	.027
....		16	.051
....		24	.074
....		31	Broke
Experiment No. 6............	.290	8	.030
....		16	.051
....		24	.074
....		28	Broke

Experiment No. 7, with a thickness of .290. This strip was loaded with 20 pounds and left undisturbed fifty-two hours. The weight being removed, the strip recovered its straightness completely.

	Thickness, inch.	Weight, lbs.	Deflection, inch.
Experiment No. 8............	.320	16	.043
....		24	.058
....		30	Broke in one minute.
Experiment No. 9............	.300	16	.043
....		24	.056
....		30	Broke after five minutes.

Experiment No. 10 was the same strip as No. 7. After the first test the load was increased to 30 pounds, after five minutes to 36 pounds, and after eight minutes to 38 pounds, when it broke immediately.

The regular increase of deflection seems to show no limit of elasticity below the breaking weight. Tests with the polariscope verified this conclusion. No. 7 had suffered no molecular change after a continuous strain of more than half its breaking weight for fifty-two hours.

If C be called the co-efficient of rupture in the formula $W = \frac{cbd^2}{l}$ Then in experiment No. 1, 4 176 pounds; in No. 2, 4 452 pounds; in No. 3, 3 552 pounds; in No. 4, 2 412 pounds; in No. 5, 3 204 pounds; in No.[6, 3 000 pounds; in Nos. 7 and 10, 4 068 pounds; in No. 8, 2 640 pounds; and in No. 9, 3 000 pounds.

If the formula $\frac{1}{8} Wl = \frac{1}{6} Rbd^2$ be used, employing the assumption that the neutral plane or axis is at half the depth, then R, or the rupturing stress per unit of section, would in each of the above cases $= \frac{3}{4}$ c. Trautwine's formula, deduced from the Millville experiments, gives for the breaking weight $W = \frac{2\,000\,bd^2}{l}$, which is a somewhat lower value than would be deduced from the above results.

DISCUSSION.

JOHN C. TRAUTWINE, Jr., Assoc. Am. Soc. C. E.—In comparing Mr. Plympton's results with that obtained by my father in July, 1868, with Millville flooring glass, and quoted by Mr. Plympton in his concluding paragraph, it is important to take into consideration the effect of the time during which the test-piece is subjected to its load. The values of the co-efficient C obtained from Mr. Plympton's experiments are as follows:

No. of Experiment.	Co-efficient: $C = \frac{\text{load x span}}{\text{depth}^2 \text{ x breadth.}}$	Time Under Final Load.
1	4172	
2	4444 (maximum).	
3	3554	
4	2406 (minimum).	4 hours 20 minutes.
5	3206	
6	2996	
7	....	
8	2637	1 minute.
9	3000	5 minutes.
10	4067	

The slight differences between Mr. Plympton's values of C and my own, as given in the above table (both deduced from his experiments),

are doubtless due to his neglecting final decimals in the quantity d^2. In experiment No. 7 the bar was loaded with 20 pounds and left undisturbed for fifty-two hours. It received no permanent set, and was again used in experiment No. 10.

It will be noticed that in the three experiments (Nos. 4, 8 and 9) in which the time under load is stated, the value of C is distinctly lower than in any of the others (in which we may assume that rupture immediately followed the application of the final load), except No. 6, which, while the least of these, is practically equal to the greatest of the time tests; and the least value of C in the whole series (2406) is given by experiment No. 4, where the time under load was greatest (four hours twenty minutes). The mean value of C for the three-time tests (experiments Nos. 4, 8 and 9) is 2681, while that for the six others (Nos. 1, 2, 3, 5, 6 and 10) is 3740, or 39.5 per cent. greater. Now, by reference to my father's notebook, I find that his test-piece, which gives $C = 12 \times 170 = 2040$, "bore load for nearly one minute before breaking," and his experiment would therefore be comparable with Mr. Plympton's Nos. 4, 8 and 9 (mean $C = 2681$) rather than with the others (mean $C = 3740$), and would thus make a better showing for the Millville flooring glass of that early day, as compared with the modern Dutch plate window glass, than would appear from a comparison with Mr. Plympton's results as a whole.

It would have been interesting if Mr. Plympton had proceeded to explain the application of his valuable results to the actual problem in hand, viz., "the pressure which plate glass windows would safely withstand when exposed to strong winds." In that case we have a rectangular plate, fastened along all four (?) of its edges and subjected to a load (approximately) uniform, but quite indeterminate and beyond our control; while in the tests we have a beam merely supported at its two ends and bearing an accurately known center load. Will Mr. Plympton give us the steps by which the safe thickness in the former case is deduced from the breaking load in the latter?

Prof. G. W. Plympton, M. Am. Soc. C. E.—In reply to Mr. Trautwine's remarks, I have only to say that he is quite right in assuming that in the majority of the experiments the rupture followed immediately upon the application of the final increment of load.

In reply to the query regarding the application of these results to the solution of the final problem (viz., the wind pressure which plate glass windows will safely resist), it may be said that nothing more is to be claimed for these results than that a safe limit for a known distributed pressure is fairly well indicated, at least when supported at opposite sides (or ends). How much this estimated thickness can be reduced for rectangular plates supported or held in water-tight fastenings along all the edges, can only be properly determined by experiments tried under such conditions.

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A CHEAP COVERED RESERVOIR.

By ARTHUR D. FOOTE, M. Am. Soc. C. E.

WITH DISCUSSION.

The domestic cold water supply for Boise City, Idaho, is obtained from a number of shallow artesian wells sunk in the hollow of the flank of the mountain which rises north of the town. The words domestic and cold are used advisedly as the town is supplied with water for irrigation purposes by canals from the river, and hot water from artesian wells a short distance east of the town will, it is said, soon be distributed by pipes throughout the city. The cold water wells overflow at an elevation of about 175 feet above the town, and at present supply about 1 000 000 gallons per day. The formation of the country in their immediate vicinity is a soft sandstone, lying horizontally, with occasional strata of talc and hard shale. The surface has the usual ridges and hollows of a rapidly sloping mountain side. Several small tunnels have been driven into the hill, in this vicinity, in a vain search for coal. An examination of the material encountered in running these tunnels suggested to me the idea of excavating a reservoir in the rock. As a preliminary experiment the sides and roof of one of the tunnels were plastered with cement mortar, which remained perfectly sound after three months.

A point was selected on the side of one of the ridges forming the gulch in which the wells are located, and slightly below them, and an open cut 8 feet wide was driven horizontally into the hill until the face of the cut was about 12 feet high. A stratum of stiff pipe-clay about 3 feet thick showed across the face of the cut, the bottom of which was 7 feet above the flow of the cut. This material was so hard and solid that it was decided to use it for the roof of the chamber, instead of excavating an arched roof, as first intended. A gallery 6 feet wide and 7 feet high was driven 10 feet, and then widened 7 feet on each side, and continued inward until there had been excavated a chamber 125 feet long, 20 feet wide, and 7 feet high. The material encountered was sandstone, easily and rapidly broken down with a pick, and was removed with the ordinary grading dump carts and horses, at a cost, including haul, of about thirty-five cents per yard.

No crack or sign of settlement or movement of any kind appeared during the excavating. It was thought advisable, however, by the directors to use every precaution possible against accident; accordingly a row of brick pillars, 8 feet from center to center, extending lengthwise through the middle of the chamber, were put in. These pillars were 20 inches square, set on flat stones 3½ feet square, with cap-stones of the same size. The cap-stones were covered with a layer of cement mortar and forced up against the roof with jacks and then the brick piers built up to them as solidly as possible. After the completion of the pillars the inside of the chamber was carefully plastered with cement mortar composed of three parts sand to one of Portland cement. The walls and roof were covered about half an inch thick and the floor about 1½ inches. A brick wall, in which were built the inlet and outlet pipes and a ventilating manhole, closed the entrance, and the water was admitted. This was about four months ago, and as yet there has not been the slightest sign of cracking or scaling of the cement, nor has there been any movement or settlement whatever in or around the structure.

This reservoir is a small affair, and was largely experimental, but the result has been that the company will now, as occasion requires, construct similar chambers on either side of the present one, greatly increasing the length, but not increasing the span of the roof. It is evident that these chambers will be much cheaper, will require less repairs, and will keep the water in better condition than any structure that could be

built outside. Of course, it is well known there are many reservoirs or "cisterns hewn out of the rock" in the far East, but I think they are somewhat uncommon in this country, and I send this description thinking it may be suggestive to any member of the Society who may happen upon similar conditions.

DISCUSSION.

A. Fteley, Vice-President Am. Soc. C. E.—The subject is an interesting one. It seems to me it should lead to some remarks from the gentlemen interested in storage of water. I might say in connection with it that a short while ago as I was making inquiries as to the influence of covered reservoirs on the purity of water supplies, I came across one fact of some interest. An engineer in California was very much troubled by an extensive growth of algæ, and having read that when the reservoirs were covered the water could be kept free from this growth, he conceived the idea of surrounding the part of that reservoir near the intake with a large floating platform. As a result he found that although there was a constant motion of the water from the outside to the portion which was covered, yet the water supplied was very much improved. This information came from a reliable source, and I thought the incident might have some interest, as I never heard of it before.

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A SIMPLE DIAGRAM, GIVING, BY INSPECTION, THE DIMENSION OF WOODEN BEAMS FOR A GIVEN SPAN AND LOAD.

By J. M. MICHAELSON (Student).
Presented by E. A. FUERTES, M. Am. Soc. C. E.

The frequent need of designing beams within the limits of the commercial sizes of timber makes the following diagram devised by the author particularly useful, as it gives at a glance, within wide limits, all the dimensions of beams which will safely bear a given load over a given span. It is the graphical expression of the equation for safe loading for a stress of 900 pounds per square inch at the outer fibers, or for any other safe load.

Before giving the construction and proof of the diagram, its use will be illustrated by an example. Let the given span be 12 feet, and the load a uniform one of 1 400 pounds per foot.

On the line AC there are given a series of figures which represent the loads in pounds per lineal foot of beam, divided by 100 ; and also widths of beams in inches. On the inclined line AB are given spans in feet. To apply the diagram we first find the point d where a load line, $14d$, parallel to the line AB intersects the span line $12c$ drawn from C through the span 12 feet on line AB. Now this line de intersects two sets of lines parallel respectively to AB and to AC, and any span line

and load line intersecting on *de* or below it will indicate dimensions of a stick which will safely bear the load. For example, we find 8×16 inches intersecting in *f*, also 9×15, 12×13, 10×15, 11×14, etc.

As a general rule, of course, the deeper a beam the more economical it is, but the selection may be governed by the method of support, the provisions against buckling, market sizes, etc.

The foregoing example involved a uniformly distributed load; but the diagram may be used for any manner of loading and for any safe stress; for a uniform load may be found which will produce a maximum stress equal to that produced by any other manner of loading. Thus for a load in the center of the span, the load taken in the diagram would be double the actual load; for a uniform load on a cantilever four times, and for one on the end of a cantilever eight times the actual. For more complicated cases of loading the method of moments is to be used.

It will be seen later from the equation for safe loading, that if any other stress than 900 pounds per square inch is used, it is equivalent to multiplying the load per foot by the ratio of 900 to the assumed stress; *e. g.*, for a stress of 1 200 pounds, the load per foot is multiplied by three-quarters, and the diagram used as before. In other words, if we increase the allowable stress on a stick supporting a given load, we can employ a lesser value of the product bh^2 given by the formula for the amount of the internal forces.

The construction is simplified by the fact that no moments of any kind are computed. On the line AB, the distances $A2$, $A3$, $A4$,......, are taken respectively $= 4$, 9, 16, ..., *i. e.*, $A2 = 2^2$, $A3 = 3^2$, $A4 = 4^2$........ These points are then joined to any point, C; and lines are drawn parallel with AB, dividing AC into any convenient number of parts. To facilitate explanations, let the lines converging in C be called span and depth lines, those parallel with AB, load and width lines, and those parallel with AC, moment lines. For a beam, with simple end supports, bearing a uniform load of w pounds per foot over a span of l feet, the maximum moment in foot-pounds is $M = \frac{1}{8} WL^2$.

The load remaining constant, the maximum moment varies directly with the square of the span, and since the distances $A1$, $A2$, $A3$, etc., on the line AB are made proportional in length to the squares of the numbers 1, 2, 3, etc., they represent graphically the moments of a constant load.

PLATE XLI.
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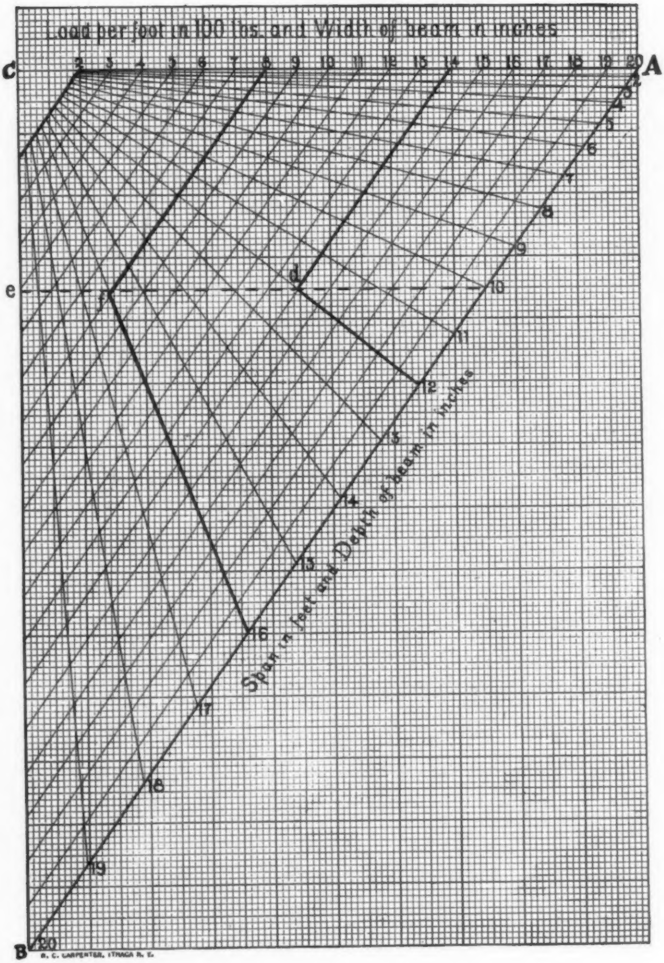
VOL. XXV, NO. 495.
MICHAELSON ON DIMENSIONS OF WOODEN BEAMS.



Again, if the span remains constant, the maximum moment varies directly with the load per foot. Therefore, for a given span and load, the maximum moment is represented by the part of the load line intercepted by the line AC, and the intersection of the given span and load lines, *e. g.*, $A12$ on AB represents the maximum moment for a span of 12 feet and a load of 2 000 pounds per foot; while $14d$, the portion of the load line 1 400 intercepted between AC, and the intersection of the span line 12 with the load line 1 400 represent the maximum moment for a span of 12 feet and a load of 1 400 pounds per foot; for, from similar triangles, $14d : A12 : : 14 : 20$, *i. e.*, $14d$ and $A12$ are proportional to the loads 1 400 and 2 000, and the span remains the same. It is then evident that all span and load lines intersecting in the same moment line have the same maximum moment.

The well known formula for the moment of the internal forces in a homogeneous prismatic beam, rectangular in cross-section is $M = \frac{1}{6}\, pbh^2$, b representing the width and h the the depth of the beam. If now p equals a safe stress of 900 pounds per square inch, and b and h are in inches, the maximum moment for safety $= 150\, bh^2$ inch pounds; and if w is the uniform load per foot in pounds, and l the span in feet, the equation for safe loading is

$$150\, bh^2 = 12\, (\tfrac{1}{8}\, wl^2) = 150 \left(\frac{w}{100}\right) l^2.$$

Thus for a stress of 900 pounds per square inch, if the load per foot be expressed in units of 100 pounds each, the span in feet, and the width and depth in inches, then the moments of the external and internal stresses have the same constant, 150; and if $b = \frac{w}{100}$, then $h = l$; and conversely. Hence the equation for safe loading is satisfied by a width and depth in inches equal numerically to the load per foot in units of 100 pounds, and the span in feet respectively. But, as shown, all width and length lines intersecting in the same moment line have the same moment. Therefore, the moment line determined by the intersection of a given span and load line contains the intersections of all width and depth lines which satisfy the equation for safe loading.

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THE ADVANTAGES OF A LONGITUDINAL BEARING SYSTEM FOR RAILWAY TRACKS.

By THOMAS C. CLARKE, M. Am. Soc. C. E.

It is now admitted by civil engineers and other railway experts, that our American system of track which has done so well in the past, owing to its strength, elasticity, and economy of first cost and maintenance, is now in a bad way, and must be modified in some manner to meet the ever-increasing weight of cars and engines. This has grown during the last fifteen years from 5 tons on a locomotive driver to 9½ tons; and 3 tons per car wheel to 5½ tons; and the end is not yet. Our track is giving out at all points. Joints which did well under lighter loads, now sink and allow rail ends to be battered down. Steel rails are not hard enough. Their heads crush, or the material itself flows under the heavy wheel loads.

A new danger has shown itself. The old form of spike can no longer hold the rails to the ties. The sinking at the joints makes the rails saw up and down, and loosens the spikes. Then the gauge spreads, and the trains leave the track. This form of accident is becoming more alarmingly frequent every year.

The supply of timber for ties is becoming a serious question. Statistics show that from 60 000 000 to 70 000 000 of wooden ties are yearly required in the United States. Chemical preparation of the timber is claimed to double the life of the tie, so far as decay is concerned, at an increase of only about one-third of the first cost. But a new trouble appears. The great weights on the rails, aided by the sawing motion above described, cut their bases into the ties, and destroy them sooner than they rot.

Thus we have rails, fastenings and ties, all failing together. What is the remedy? Mr. Whittemore, of the M. and St. P. R. R., proposes to increase the bearing surface of rails by making the heads flat and increasing their width. Mr. Sandberg, of London, sees no other way but to increase all the dimensions of the present rail and its fastenings, until his new "Goliath" pattern weighs 100 pounds per yard. Improved forms of joints—notably the Fisher, Cloud & Thompson, so called from their inventors, have met with considerable favor.

Cutting into the ties can be prevented by the use of metallic tie-plates. This, while excellent in itself, complicates the problem of fastening the rails and ties together. In England, the cast-iron chair on every tie is a good but expensive remedy.

Various improved forms of spikes and bolts have been devised to hold the rails and ties more firmly together. The most radical change, however, is the adoption of steel cross-ties, originally devised for countries where timber was scarce, or where wooden ties were eaten by insects. They have spread all over Europe, and are now being used experimentally on some of our larger railways. Besides their durability, they have the merit of allowing the rails to be firmly fastened to them and spreading of rails is prevented. But the sinking of the rail joints still causes the rail to saw up and down. As it cannot leave the tie, they both move together, and after a time get loose in the ballast. Also, the tie, being but a shallow piece of channel iron with flanges 2 to $3\frac{1}{2}$ inches deep, has no stiffness crosswise. The wheel is only supported by a short length of the tie, and they bend down under the rails, assuming a shape with two bends downward and one upward in the middle. (See Fig. 1, Plate LII.) This works the ties loose in the ballast, and from these causes, it is difficult to keep a metallic cross-tie system in good line and level.

From this smallness of the bearing surface, the metallic cross-tie track, under American wheel loads at least, not only sinks at the joints

but at each tie as it is passed over, making an uneven track longitudinally. There is also a serious complaint of the want of elasticity in this form of track. The first cost of these ties is considerable, as will be seen from the comparative table given hereafter. Various attempts have been made to improve them, by inserting wooden blocks, etc. This only leads to complication of parts and increased cost, while it does not remedy the vital defect of weakness of joints, and the consequent vertical movement of the rails.

On considering all these matters, and personally examining a great many improved forms of track, it seems to the author that the only radical cure of the difficulty is to return to the old form of continuous rail and continuous bearing, improved to avoid those defects which experience has shown, and made capable of being extended in dimensions and modified in material to meet the ever-increasing loads of the future.

The defects of the continuous-bearing system as formerly made are as follows: The early form of continuous rails was made of such soft iron that the bearing parts cut into each other and soon worked loose. This cannot occur with steel members of proper proportions. The rails rested on longitudinal timbers. These had to go down so deep that they cut off drainage between the rails. Also, it was difficult to unite them to the wooden cross-ties necessary to preserve the gauge. The great number of joints caused decay. For all these reasons the continuous system, although giving a very smooth track while new, soon got out of order, and was replaced everywhere by the wooden cross-tie system, which was less expensive to make and far less so to maintain.

The radical, and as the author believes, irremediable defect of the cross-tie system is its low joints. The rail itself is treated as a continuous girder resting on yielding supports and spliced at the joints. If the attempt is made to splice the joint within the rail depth, as by angle bars, it deflects on account of want of depth. As the joint goes down the middle of the rail goes up. If we strengthen the joint by putting pieces below it, intended to be in tension, while the rail is in compression, thus converting it into a deeper form of girder, the necessity of allowing for expansion and contraction makes it necessary to use oval holes for the bolts. These allow the bolts to move longitudinally, and thus they do not resist the shearing strains, and hence do not prevent deflection.

This form of joint works well while new, but after a while gets loose like the others, and low joints are found. The only way, in the author's belief, to absolutely prevent low joints is to place continuous bearing pieces under the whole length of the rails and breaking joint with them. The solid part of the bearer supports the joint of the rail, and the solid part of the rail bridges over the joint of the bearer, and what is of the highest significance, a small amount of longitudinal movement or play between the upper and lower members will not cause deflection, when arranged as described. The requisites of a properly designed longitudinal system are as follows:

First.—The first is, that the longitudinal bearer under the rail shall be stiff enough to transmit the load to such a distance, on each side of the wheel, as will limit the pressure to not much over 2 tons per square foot of bearing surface, without requiring excessive width. Experience has shown that a greater pressure than 2 tons per square foot will sink ties too deep into the gravel or broken stone.

Second.—The next thing is to attach the rails and bearers together by a form of fastening, strong enough to resist all strains and shocks, and yet allow of freedom of the rail to expand and contract, independently of its bearer. It must also be held to its bearer, so that creeping of the rail on the bearer may be prevented, and that without any notching or cutting of the rail that will impair its strength. The rails must break joint with the bearers. The fastenings must be so made that the rails can be quickly removed and replaced by new ones without disturbing the bearers. The fastenings must be able to hold for a time a broken rail, so that it will safely pass the trains, and no system but the longitudinal can do this.

Third.—The bearers (and rails) should be united firmly together by light metallic gauge ties, placed near enough to properly preserve the accuracy of the gauge.

Fourth.—The bearers and gauge ties should be of such shapes as can be easily tamped with gravel or broken stone; as will stay in place vertically, laterally and longitudinally, and will allow of drainage to pass between them.

Fifth.—The system should be so planned that no difficulty of construction can occur at curves, either in alignment or elevation of outer rail. Also, it should be so made as to easily join to the ordinary form of T-rail at turnouts and switches.

Sixth.—Besides the obvious advantages which such a construction gives, there are two others: The upper rail can be made of a harder and better worked steel, while the bearer can be made of a softer and tougher quality of metal. Probably basic steel would do for this.

Owing to the rails being supported under their entire length by continuous bearers, they can be made of less depth and sectional area in their lower flanges than at present. The metal so saved can be put into the head of the rail, where it is most needed.

It is believed that rails can be designed for a longitudinal system with heads 3 inches wide, and instead of weighing 110 pounds to the yard, as in the last sheet of sections given by our Committee on forms of rails, they need not weigh over 70 pounds to the yard. This saving of metal can be applied to reducing the cost of the whole system. The wear being confined to the upper rail, the amount of metal which goes into the scrap heap is the least possible. Longitudinal bearing systems are not new, nor entirely experimental—as may be seen by reference to the report of E. E. R. Tratman, Assoc. Am. Soc. C. E., to the Department of Agriculture. This system has been in use on the Austrian State Railways, Northwestern Division, for eleven years. It is the invention of Mr. Hohenegger, Chief Engineer, from whose report, quoted by Mr. Tratman, the following description is condensed. (See Figs. 2 and 3, Plate LXII.)

There are 59½ miles, of which 37.21 miles are tangents, and 22.30 miles curves varying from 9 840 to 935 feet radius. There are daily, both ways, two fast express, two express, two slow passenger and ten freight trains. Passenger engines weigh 42 tons with 6.2 tons on driving wheels. Freight, 45 tons with 5.67 tons on driving wheels. Only one longitudinal bearer has been renewed. The only wear observed has been where, in some cases, the flange of the inner rail on curves has worn slightly into the bearer. The longitudinals, like the rails, are laid just as they come from the rolls, but no rusting at all has taken place. The track is reputed to stand admirably and requires little labor for maintenance, either on tangents or curves. There is no trouble with the fastening of rails to the bearers, and the nuts being held in position by nut-locks, tightening of the nuts is rarely necessary.

The fastening of the rails to the bearers prevents creeping of the rail on the longitudinal by the firm hold of the clamp on the rail flange and on the rib of the longitudinal. The inclined outer face of the clamp admits of

an adjustment of the gauge by slacking one nut which allows the clamp to rise and move back. The rail is then shifted, the opposite clamp pushed down to fit and both nuts screwed tight. The rail flange butts against a boss on the lower side of the clamp which receives the force of the lateral thrust from passing trains, and transfers it to the rib of the longitudinal, thus protecting the bolt from wear. As there are no spikes to be driven, a smaller gang can keep the track in order. In consequence of the continuous support of the rails no rails nor splice plates have ever broken. The rail weighs only 64.30 pounds per yard, while a rail on cross-ties would have required a weight of 100.43 pounds per yard.

Obviously there are two defects in this design of track. The form of the longitudinal is such that Mr. Hohenegger tells us that the ballast which consists of broken stone and river gravel, is compressed by tamping and the pressure of passing trains, to such an extent that it can only be loosened with a pick. This compression extends a foot below the longitudinal and prevents the quick drainage of water from between the rails. This could have been prevented by a better section of bearer, possibly of a T shape. The ballast could then be tamped against the vertical member from both sides and there being nothing to confine it there, the packing would not have taken place. Another mistake was to make the joints of the rails and of the longitudinal bearers coincide, instead of breaking joints. Mr. Hohenegger says nothing about the existence of low or battered joints, but they must have existed, and breaking joints with the bearers would have prevented them.

A comparison of the cost of the cross-tie systems now in use, with that of a continuous bearing system is as follows, prices of material being alike:

First.—Wooden cross-tie system and 60-pound rails as used on local lines.

	Mile of Single Track.
106 tons rails at $35	$3 710
2 640 ties at 55 cents	1 452
352 splice bars and bolts	440
Spikes	130
Laying track	500
	$6 232

Second.—Improved system with 80-pound rails, treated ties, tie-plates, and two pairs interlocking bolts per rail, as used on Trunk lines.

140 tons rails at $35	$4 900
2 816 treated ties at 80 cents	2 253
352 tie plates at 10 cents	35
352 long splice bars and bolts	720
Interlocking bolts	231
Laying track	600
	$8 739

Third.—System further improved by use of 85-pound rails, and steel cross-ties.

149 tons rails at $35	$5 215
2 464 steel ties and fastenings at $3	7 392
352 long splice bars and bolts	720
Laying track	700
	$14 025

Fourth.—Hohenegger system of longitudinal bearings used on Austrian State Railways. Rails 64.38 pounds per yard.

113.4 tons rails at $35	$3 919
105 tons longitudinals at $46	4 835
32 tons, cross-ties, plates, splice bars and bolts at $50	1 600
Laying track	800
	$11 154

From this it would seem that a first-class track, made with metallic longitudinal bearers, need not exceed the cost of a first-class track laid with metallic cross-ties.

A letter and drawings received from Herr Hohenegger are appended.

APPENDIX.

In answer to your esteemed query, dated June 17th, I beg to give the following information, taking reference to the "Advance Copy" received from you.

Article 1. I have used in my system of superstructure with longitudinal bearers that is discussed by you, both the combined joint as represented in your sketch and the broken joint.

From the drawings embodied, you will understand the construction of the broken joint, in which the joints of tie and rail do not coincide. The combined joint in which rail and longitudinal bearer butt against each other on the mutual saddle-shaped splice, *S*, facilitates the laying of the superstructure somewhat, and gives generally good results, because the carrying capacity of rail angle bars and of tie splice equals about that of rail and of longitudinal bearer; the broken joint, however, seems, in the length of time, to offer more resistance against the depression of the joints. I have tried both systems from the beginning, in order to gather experience and to reinforce, if necessary, those parts of the joints which should prove too weak. Obvious weaknesses in this regard have not been noticed up to the present time, but I would now give a larger bearing surface to the tie-saddles.

Article 2. The fastening of the rails on the longitudinal bearers by means of wedge-shaped tightening plates, which bear against wedge-shaped ribs of the longitudinal bearers, proves eminently satisfactory, with the only alteration, that we used two-sided unsymmetrically formed tightening plates instead of the one-sided ones, to facilitate the obtaining of the correct gauge where the cross-ties are not well punched or where they are bent as in curves.

The wedge-shaped form of the tightening plates has done away entirely with the very undesirable rusting together of the bolts with the bearers. The exchanging of worn-out or injured rails is effected very easily without any disturbance whatsoever of the position and of the fastenings of the longitudinal bearer.

Within the fourteen years of the existence of this superstructure with longitudinal bearers, two cases only of breakage of rails have occurred, and in these several trains passed without derailment over the remnants of the rails that had been broken in many pieces. In both cases blowholes caused the heads of the Bessemer steel rails to be shattered in many small pieces, so that the wheels ran without being derailed, partly on the rail-web, partly on the longitudinal bearer, though one of the broken pieces belonged to the outer rail of a rather short curve.

Article 3. I have never introduced gauging rods proper, as they are used in the superstructure with longitudinal bearers of the German rail-

ways, but have been satisfied with one transverse angle for each 9.84 feet (3 meters) of bearers fastened to the lower side of the bearers with short pieces on saddles, *s*. This transverse connection has perfectly sufficed to maintain the gauge; if it should not prove sufficient more cross-connections would be put in place without great expense, but they have not been necessary so far.

Article 4. Longitudinal bearer, transverse angle-tie and tie-saddles are easily tamped with gravel when the superstructure is newly laid; in later years, after the box-shaped volume of ballast underneath of the longitudinal ties has consolidated into a mass of beton, the longitudinal bearers must be lifted off it and the beton-like old ballast has to be replenished by a layer of fine ballast. However, this work becomes necessary only very seldom because the superstructure with longitudinal bearers retains its position much better than if laid with transverse ties, which each give way separately.

The (lifting) raising of the superstructure when depressed, is more difficult with sunk iron transverse ties than with longitudinal bearers, for the following reason. It is not possible to raise a single transverse tie without unscrewing a pair of rails, whereas with longitudinal bearers rail and bearer can be lifted together, necessitating only the loosening of the rail angle bars.

Article 5. The T-shaped bearer, as suggested by you, was tried in 1873 by the "Rechts Rheinische Railroad," but has not proved a success, because the vertical web of the T tears asunder the box-shaped mass of beton, instead of holding it together and of gradually rendering it a unit. All longitudinal bearers and transverse ties shaped similarly have been discarded for some time as non-practical. In order to lay the curves of the superstructure, the longitudinal bearers have to be bent in the ironworks according to the requirements of the earth curve. The supervising engineer must have an accurate table that shows how the segments follow each other, and he must lay his longitudinal bearers consecutively and in accordance with it. The laying is done very rapidly if this rule be observed. Up to 984 feet (300 m.) of superstructure are put down per diem on track where frequent trains are running. That is twice as much as could be exchanged of superstructure with transverse ties. The superelevation of the outer rail does not offer any known difficulties. The connection with cross-overs (Weichenstrasse) and switches, is not more difficult than in superstructure with transverse ties, if rail angle bars of equal section are used and if the cross-overs are put down on transverse ties. We do not have cross-overs on longitudinal bearers.

Article 6. The rails on longitudinal bearers are essentially lighter than those on transverse ties. They weigh only 18.106 pounds per foot (27 kg. per meter), against 22.132 pounds per foot (33 kg. per meter). Transverse fractures of rails have not yet occurred, those two cases excepted where the rails were destroyed through blowholes of the ingots.

PLATE XLII.
TRANS. AM. SOC. C. E.
VOL. XXV, NO. 496.
CLARKE ON BEARING FOR RAILWAY TRACKS.

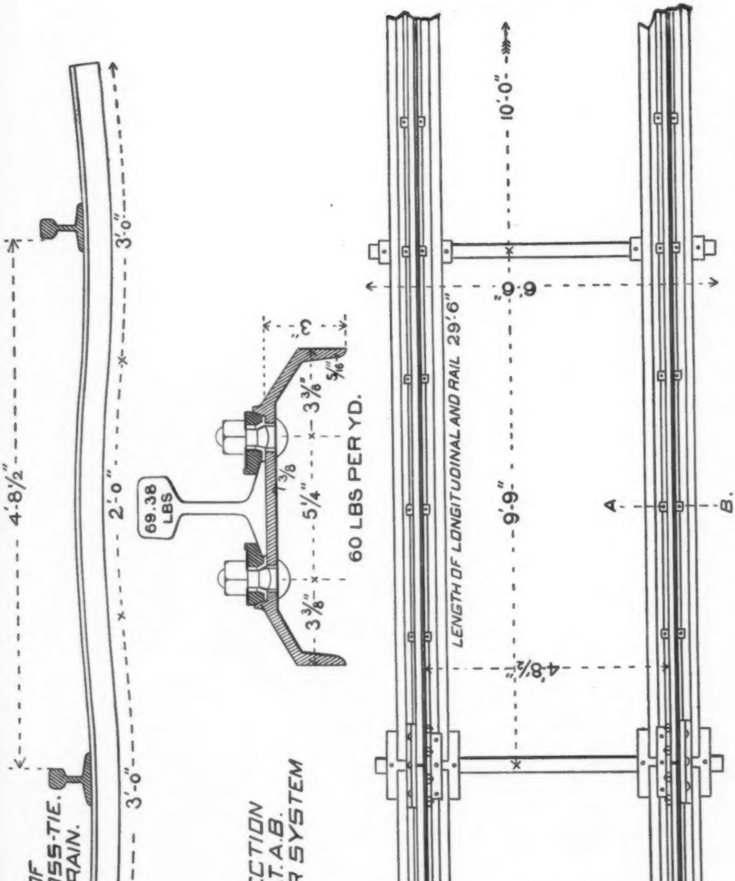


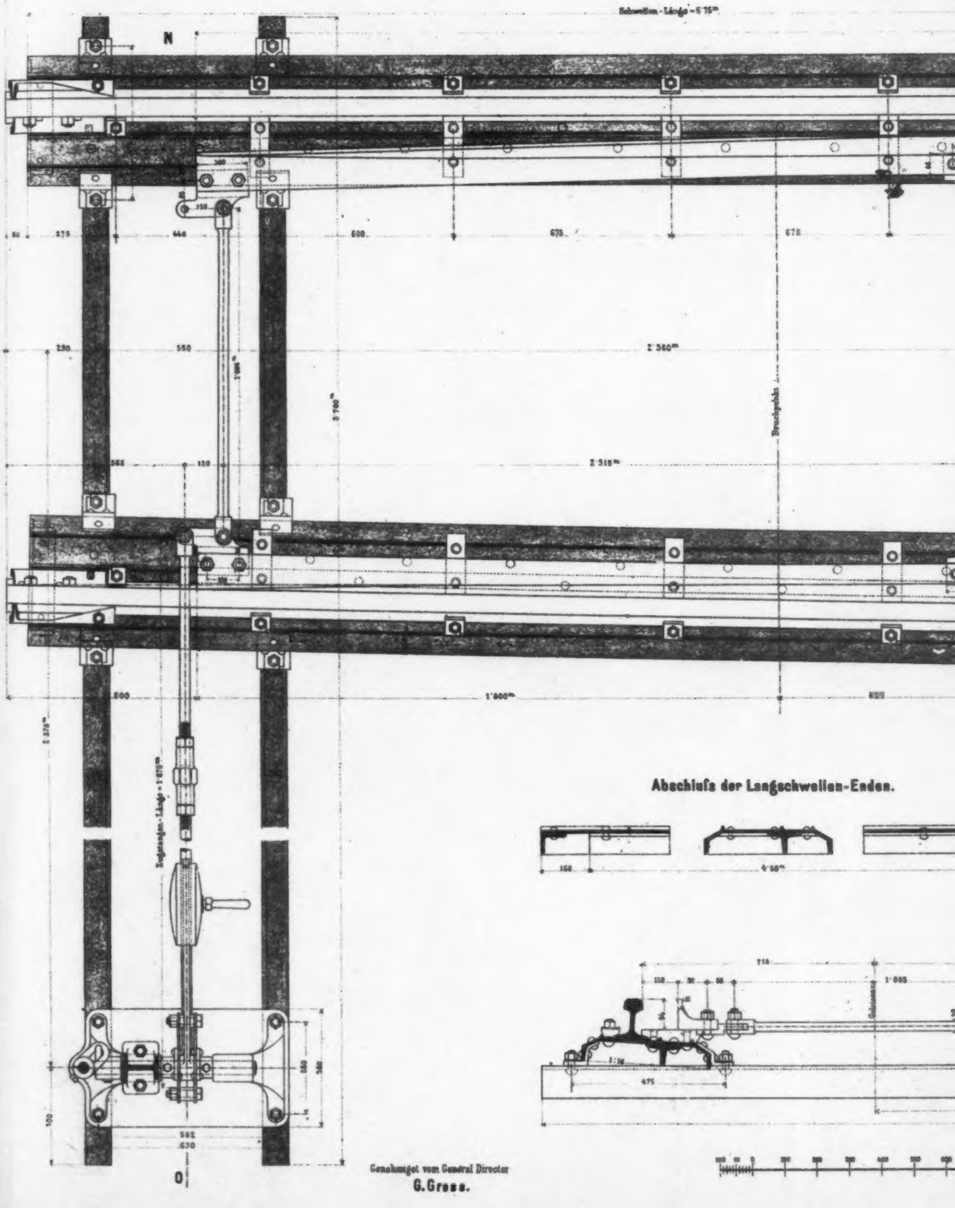
FIG. No 1. BENDING OF METALLIC CROSS-TIE. UNDER TRAIN.

FIG. No 2. CROSS SECTION OF FIG. 3 AT. A.B. HOHENEGGER SYSTEM

FIG. No 3. PLAN. DO. DO.

OESTERREIC

EINFACHER WE



Genehmigt vom General Director
G. Gross.

OESTERREICHISCHE NORDWESTBAHN

EINFACHER WECHSEL MIT LANGSCHWELLEN.

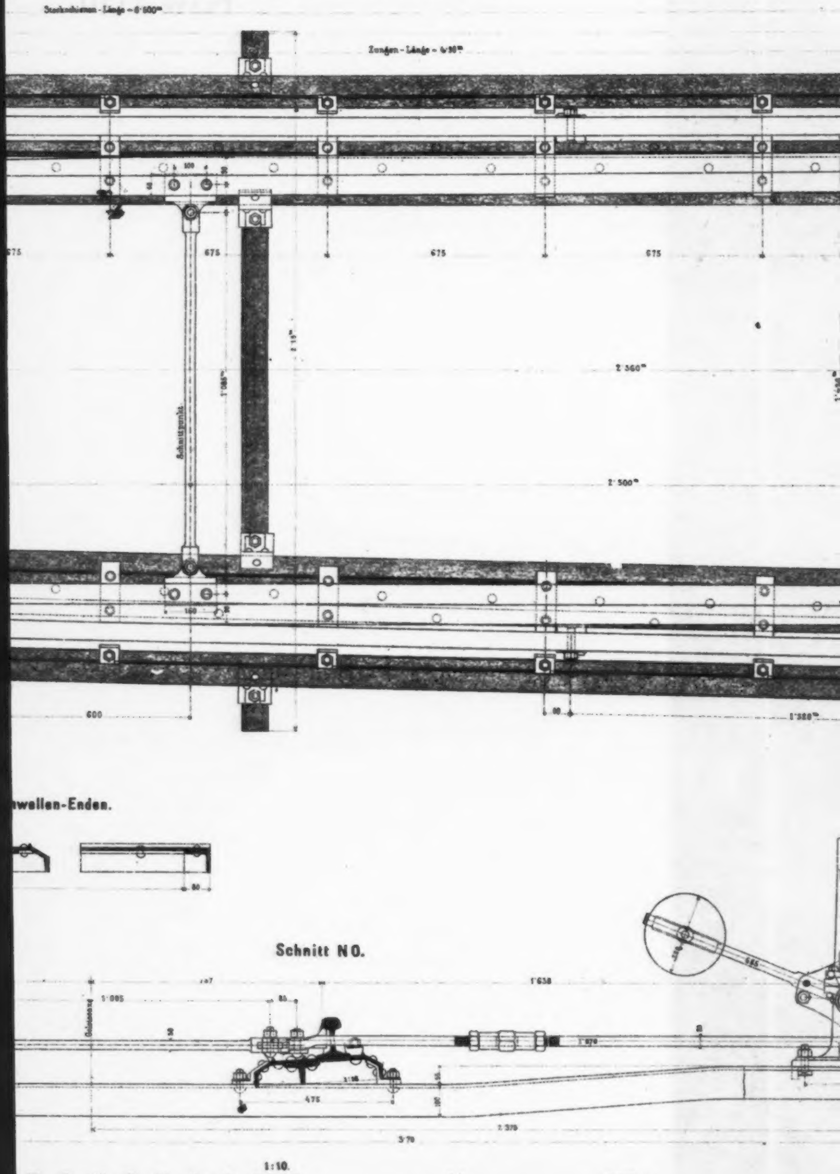


PLATE XLIII.

~~PLATE LIII~~

TRANS. AM. SOC. CIV. ENG'RS
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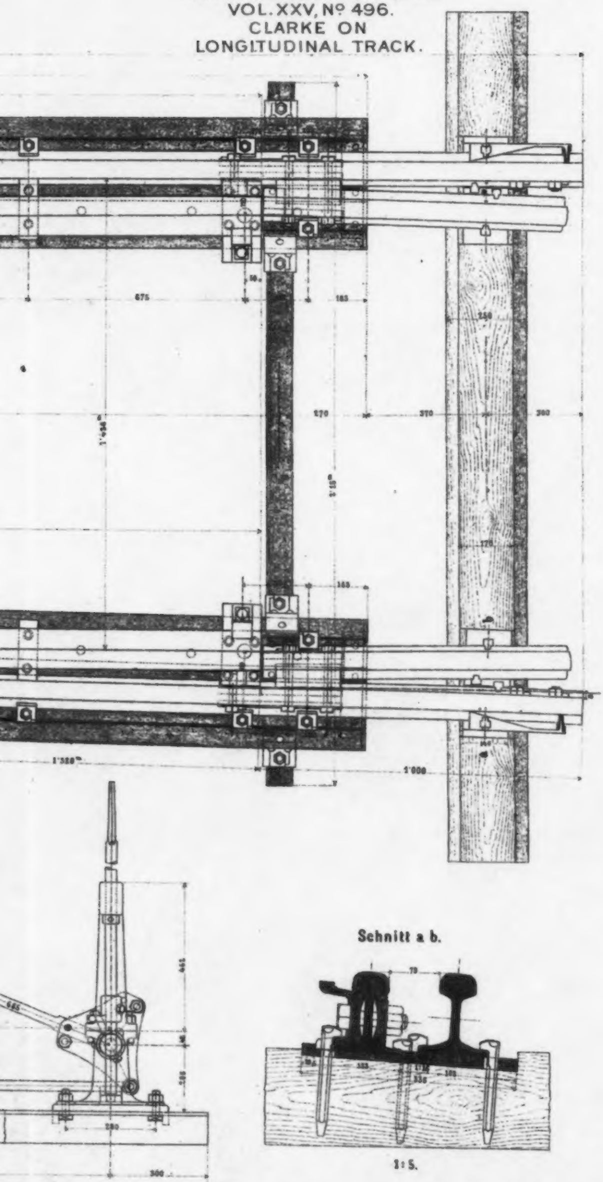
CLARKE ON
LONGITUDINAL TRACK.

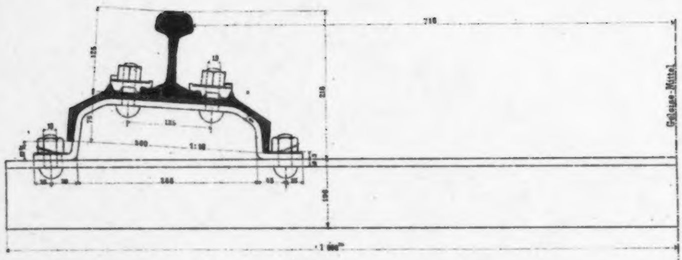


Wien im November 1883.
Angefertiget vom Baudirektor
W. Hohenegger.

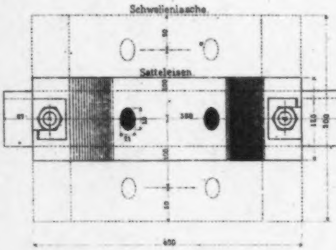
OESTER

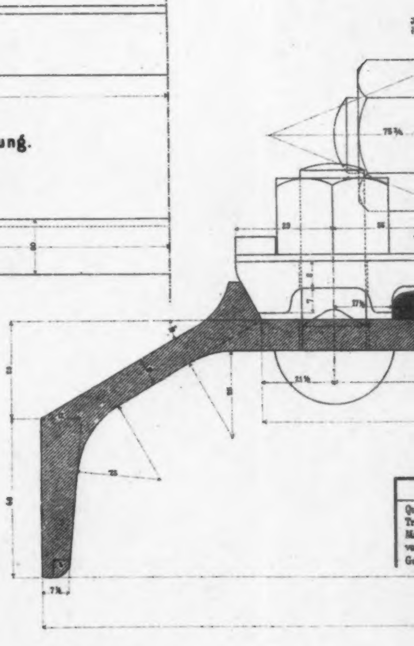
EISERNER OBERBA

System V.



Querverbindung.

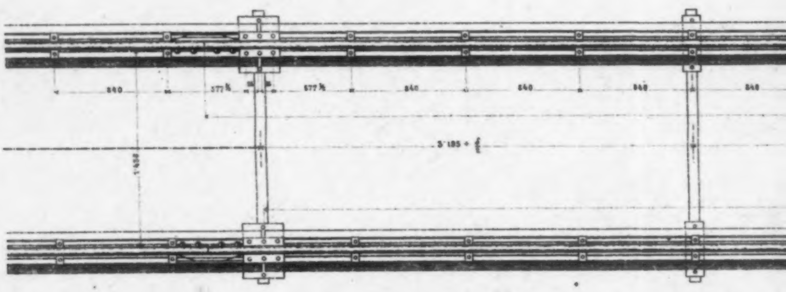




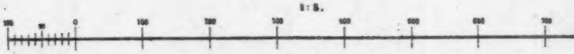
Materialbedarf.

Stück	Gegenstand.	Gewicht in Kilogramm. einzeln	Gewicht in Kilogramm. zusam.	Stück	Gegenstand.	Gewicht in Kilogramm. einzeln	Gewicht in Kilogramm. zusam.
2	Stahlschienen à 9 m ⅟ₛ	284·70	569·40	4	Schwellenwinkel	1·45	5·80
2	Langschwellen à 9·725 ⅟ₛ	284·00	568·00	44	Schienenwinkel	0·23	10·12
3	Quer-Winkel à 2·6 m ⅟ₛ	26·40	79·20	20	Schwellenbolzen	0·32	6·40
2	Schwellenlaschen 300 %	10·30	20·60	36	Schienenbolzen	0·30	10·80
4	Satteleisen à 120 % ⅟ₛ	4·20	16·80	12	Querwinkelbolzen	0·29	3·48
2	Winkellaschen	8·17	16·34	8	Sicherheitsplättchen 22	0·08	0·64
2	Innenlaschen	4·45	8·90	68	dto. 19	0·04	2·72
8	Laschenbolzen	0·60	4·80		Gew. pr. Schienenlänge Geleis		1324·00

Gewicht pr. Mtl. Meter Geleis = 135·8 Kilogr.



1:5.



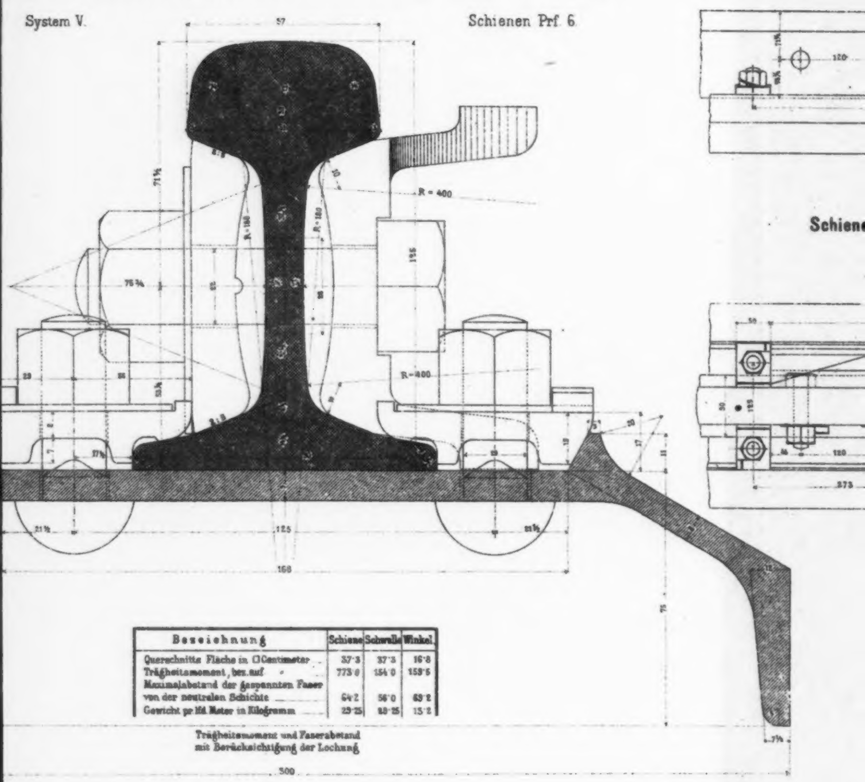
Genehmiget vom General Director
G. Gross.

OESTERREICHISCHE NORDWESTBAHN.

RNER OBERBAU MIT LANGSCHWELLEN AUS FLUSSEISEN.

System V.

Schienen Prf. 6.



Bezeichnung	Schiene	Schwelle	Winkel
Querschnitts Fläche in □Centimeter	37·3	37·3	16·8
Trägheitsmoment, bez. auf	773·0	154·0	153·5
Maximalabstand der gespannten Faser von der neutralen Schichte	64·2	56·0	63·2
Gewicht pr lfd. Meter in Kilogramm	29·25	29·25	13·2

Trägheitsmoment und Faserabstand mit Berücksichtigung der Lochung

Schiene

Geleisanlage.

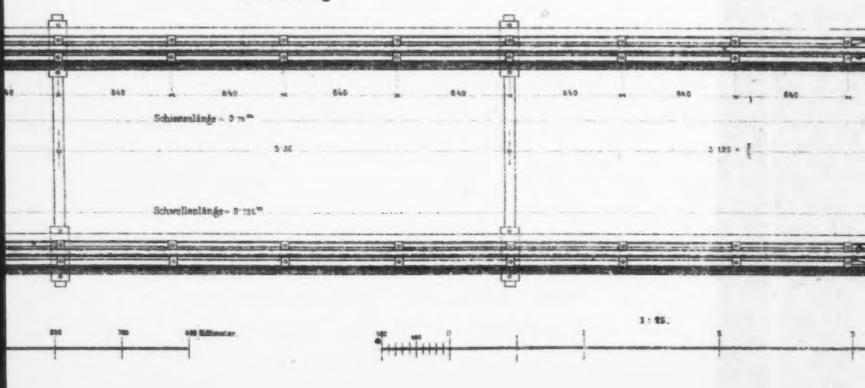


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PLATE XLIV.

TRANS. AM. SOC. CIV. ENG'RS
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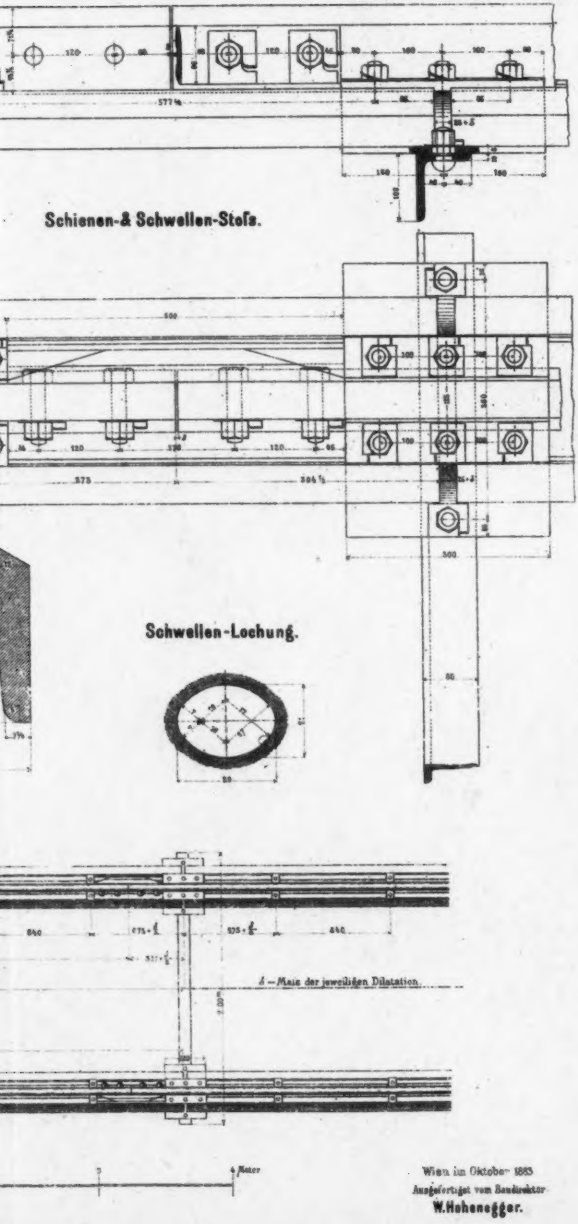
CLARKE ON
LONGITUDINAL TRACK.



Wien im Oktober 1883
Ausgefertigt vom Baudirektor
W. Hohenegger.

OESTERREICHISCHE NOR

UNTERLAGS - SPAN

für Holzschwellen - (

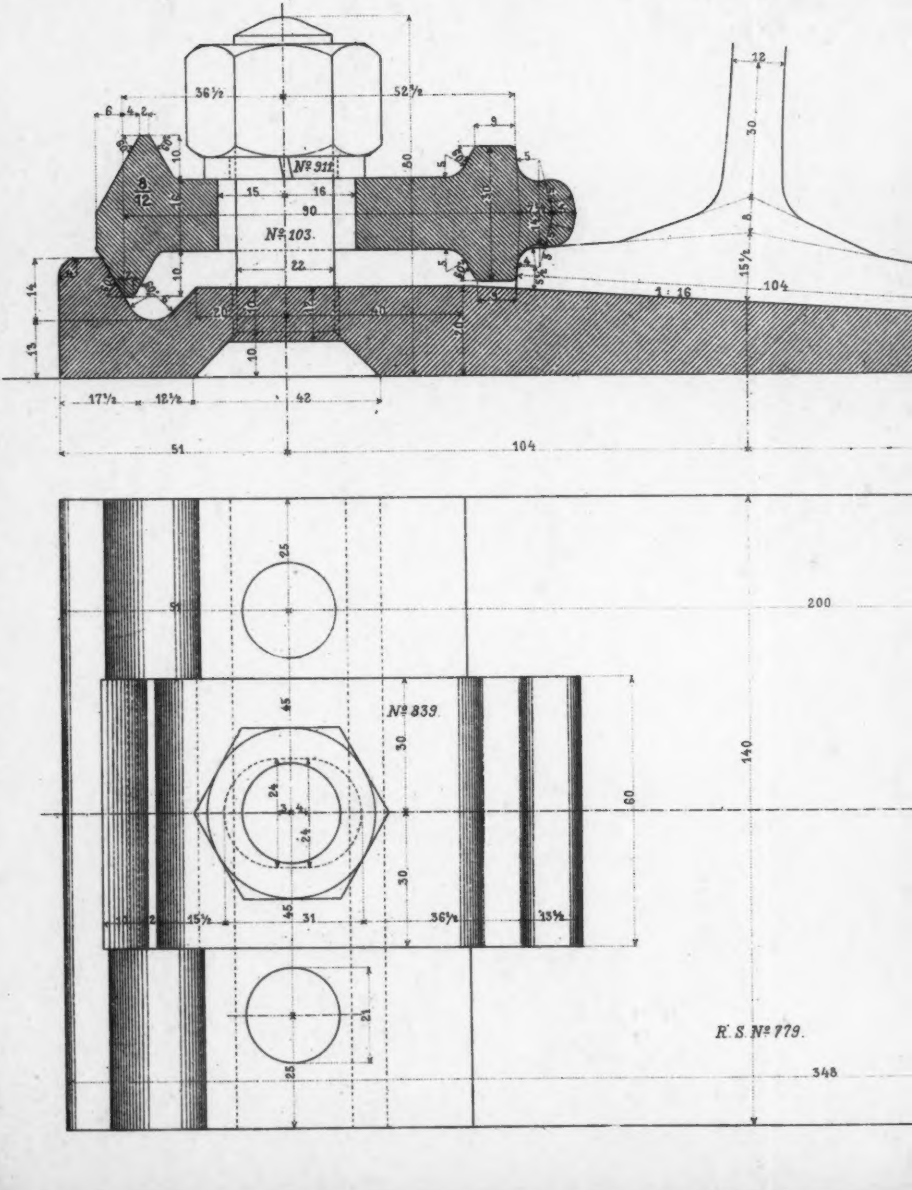


PLATE XL

TRANS. AM. SOC

VOL. XXV,

CLARKE

LONGITUDINA

HISCHE NORDWESTBAHN

GS - SPANNPLATTEN

zschwellen - Oberbau.

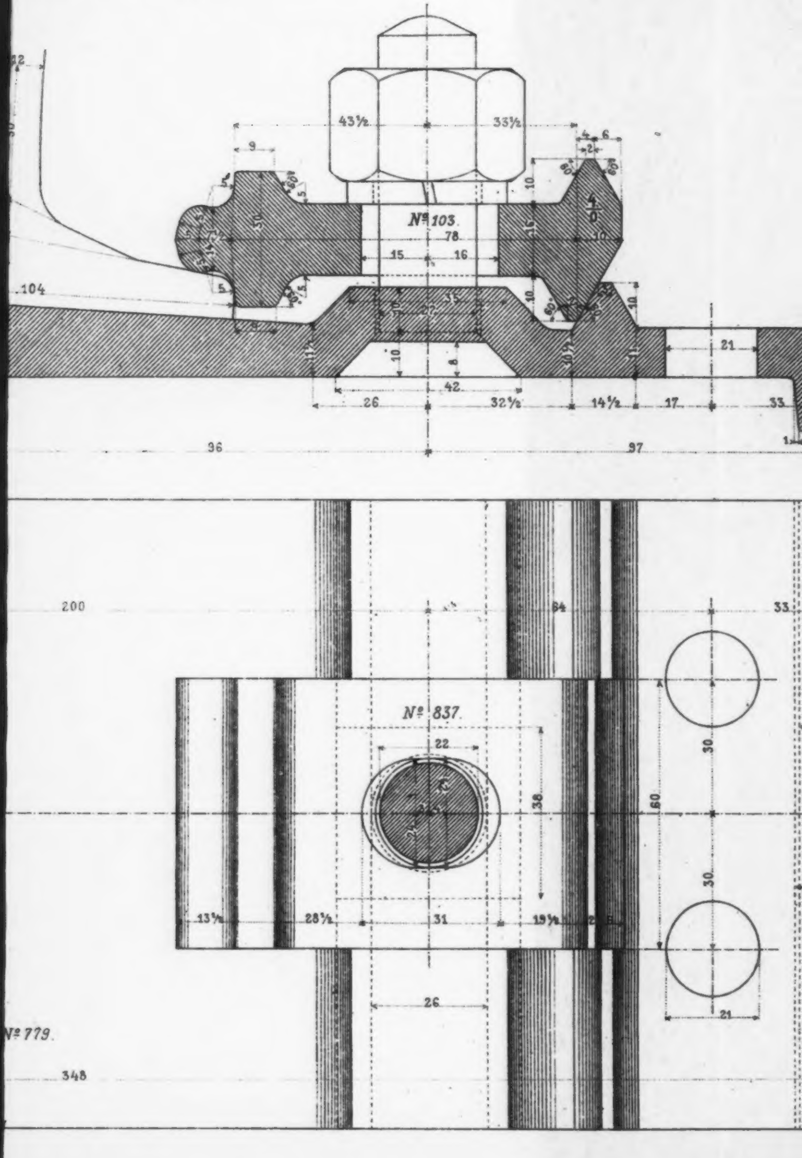
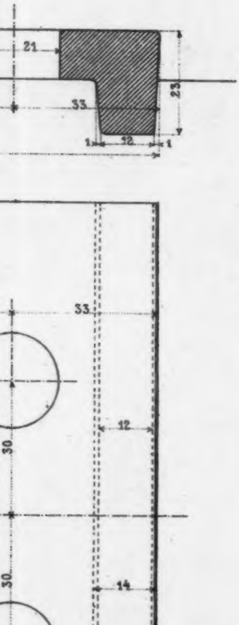


PLATE XLV.

~~PLATE LV~~

NS: AM.SOC.CIV. ENG'RS
VOL. XXV, Nº 496.
CLARKE ON
ONGITUDINAL TRACK.



The rails are, as a matter of course, made of harder steel, whilst the longitudinal bearers are manufactured from soft Gilchrist—Thomas metal.

Up to date, 55.89 miles (90 km.) about, of superstructure with longitudinal ties have been laid and have given satisfaction in every respect. The quicker extension of this system is retarded only by the high prices of the iron works. Difficulties as to the drainage of this superstructure have not appeared; coarse, water-transmittent sand must be called the most appropriate material for ballasting. The superstructure with longitudinal bearers has especially given good service with sliding substructure. Our superstructure with wooden cross-ties has been considerably strengthened and has been made steadier by the introduction of the large tie-plates of my system.

You will see the details of these plates in the drawings enclosed. (Plates LIII to LV.)

Hoping that these communications will prove sufficient,

I remain,

Respectfully,

HOHENEGGER.

DISCUSSION.

WM. P. SHINN, Past-President Am. Soc. C. E.—Mr. President, I am glad to see somebody take up this subject of "permanent way" (as our friends across the water call it) and make some suggestions in regard to an improvement in the construction of our railroad tracks. It is now fully twenty-five years since I have dared to differ with most, in fact almost all of my co-laborers in the railroad field, in regard to one of the fundamental conditions of railroad track. Almost every engineer —almost every railroad man—will say, unquestionably and unalterably, the track must be "elastic." As has been suggested, we do not make the sub-structure that is to sustain all this weight elastic. We do not try to make the foundations of an immense building, so that it will yield when the weight is brought upon it, and the idea of there being a necessity for a railroad track to be elastic is, in my opinion, erroneous. As I said, I have held that opinion for twenty-five years. I have been on the other side of the fence. Our Member, Mr. Clarke, has got one foot over. He is proposing to make the track more rigid and give it a better support. I have taken the ground that a perfect track will be upon an absolutely rigid foundation, but it will also be absolutely a plane in its structure. What we need is a support which will hold the rails to the place where they are put. Mr. Clarke in his paper dwells upon the sinking of the joints, but he does not go far enough nor mention one-half the difficulty. Any one

who will watch the passage of one of our ordinarily heavy trains over even the best of our tracks, will see that every cross-tie, as the weight of the driving wheels and the loaded trucks come upon it, sinks down frequently half an inch and rises up again. What kind of a foundation is that, to carry at a speed of 50 miles or 60 miles an hour a machine which weighs 50 or 60 tons? There is another feature in this connection which seems to have been lost sight of. We know that the amount of power required to move our trains depends very largely upon the gradient up which they have to be moved, and the maximum load of our trains is determined by the maximum gradient over which the trains have to go, yet we are constantly providing a gradient of 1 to 2 per cent. in front of the driving wheels, so that even on a level road or foundation it is all the time climbing a gradient of 2 per cent. and sometimes 3 or 4 per cent. The experiments made by Mr. P. H. Dudley, C. E., with his dynagraph show that on the New York Central and Harlem River Railroad, a passenger train weighing 378 tons, moving at 55 miles per hour, required 720 horse-power on an 80-pound rail, and 820 horse-power on a 65-pound rail, an increase of 100 horse-power, or 14 per cent. Mr. Dudley expresses the opinion that on a very stiff 105-pound rail, the difference would be 200 horse-power, or 28 per cent., which goes to show that by giving the locomotive a firm support for the track upon which it lies, not only may we save immense sums in the maintenance of track, as has been pointed out in the paper, but we may very greatly increase the economical value of our power. Now, if that is true, the amount which such corporations as our trunk lines can afford to spend in order to get a perfect and rigid track is very large. It would probably justify an expenditure of two or three times the present expenditure on the track.

I hope at a future meeting of this society to submit some views on this subject which will be a little more definite than any which I can give you to-day. I hope to submit a more complete remedy for the problem than Mr. Clarke has given. His deductions are entirely in the right direction, but do not go far enough. If any of you have ever seen the freedom from resistance and shock with which a wheel moves over a plate of glass you can realize what would be the possible advantage of having for our trains a perfectly level and perfectly rigid track; so far we feel that we have been getting along with what was perhaps a necessity under certain economical conditions, but it is not a necessity any longer. The weight of our equipment, the volume of our traffic, the speed of our trains, all demand that we should have a more complete, a more perfect "permanent way."

Henry G. Prout, M. Am. Soc. C. E.—Probably no one will differ with what has been said as to the importance of increasing the stiffness of our railroad tracks. I merely rise to say that the track proposed by Mr. Clarke is apparently a very inadequate effort towards doing this.

You will see that the bearing surface between the longitudinal which carries one rail, and the ballast is only about 30 square feet; in ordinary track we have 48 square feet under one rail. With that inadequate bearing surface, as will be quite apparent to any railroad man, you would get serious rolling of your train, because it is hardly possible that the ballast should be equally firm on both sides of the track. There will be differences in the tamping or in the drainage sufficient to make one side deflect more than the other. Further, I do not see what there is to prevent the track creeping on grades. The creeping due to wave motion would not be so great as the creeping due to expansion, but the creeping due to expansion would be very great. The stringers would not be subjected to as great variations of temperature as the rails, but they would be subjected to great variations, and there is no reason why they should not expand down hill and contract down hill on grades. These, it seems to me, are fundamental and sufficient objections to this type of track.

James B. Francis, Past-President Am. Soc. C. E.—I am not a railroad engineer at all, but my recollections go back to the first railroads, those built in the old country and those built in this country. They were all laid upon stone blocks and, without knowing positively what the reasons were for the change from stone to wood, one thing that was said, I know, was that it pounded from non-elasticity in the track—that was what was prominently stated. They very quickly passed from stone blocks to the wooden sleeper, and the wooden sleeper has the elastic motion which the other had not. Not being a railroad man I cannot go into the reasons for these things; I merely speak of facts.

A Member.—I think the difficulty there, was that it was not a continuous bearing; undoubtedly it was a sort of bearing that would give very heavy shock. This was given as one reason why the stone blocks were taken out.

P. F. Brendlinger, Am. Soc. C. E.—The proof of the pudding is in the eating of it. I am very sorry that we have not more of the details of the Austrian system; we only have here an end view, cross-section and plan. It is not even shown how the ties connect the two longitudinal stringers except in the plan. The ties appear to be spaced 9 feet 9 inches and 10 feet apart. It is easy enough to build a railroad on a tangent and keep it in alignment. But it is quite a different thing with a curve, not only in regard to the elevation but also in regard to the alignment.

I believe that it will be a great many years before we will see this system, or anything similar, used in this country where there is competition and cutting of rates all the time (except in isolated cases) until you get considerable dividends on the investments. I know if this system were used on most roads there would be no dividends in a great many years. It seems to me that a stringer, which has such a sharp

knife edge, is liable to be depressed into the road-bed to such an extent whenever the train goes over it, that a very bad surface would prevail all the time. In summer this may be overcome by raising and tamping, but in winter the usual practice of "shimming" to keep a good surface will be difficult to perform. On embankments it will be difficult to keep a good surface, as the rains wash away the material of the road-bed down one side or the other. Fig. 1 shows a bent metallic cross-tie under a train. I never heard that that was the objection to metallic cross-ties. The great objection, as I understand it, is the tendency of the ties to move laterally. I have been informed that on the New York Division of the Pennsylvania Railroad, where metallic ties were tried thoroughly, various devices were used to prevent the side or lateral movement of the ties on curves without success. I got up a plan similar to this in Pittsburgh, in 1872, except that the stringers were made of wood. It was tried and then abandoned because it was much more expensive to keep up the track, and if the track was not kept in good order all the time there would be a seasick crowd of passengers on the train. I think it will be a great many years before this system is used in this country. We will not have it as long as we have so many competitive railroads.

O. Chanute, President Am. Soc. C. E.—I would say that it is my own recollection that at an early date a track system was tried in England, in which the rails were laid upon continuous walls in lieu of the stone blocks that were first used. Mr. George Stephenson tried once to make his road something like this, but it was said that those stone walls were found so rigid that they were abandoned. Then experiments were made with longitudinal systems, sometimes with the various sandwich girder systems in which the rails were bolted between two pieces of timber. There were quite a number of those shown in the report upon permanent way, published in 1858. That also shows the pillow saddleback rail, the bottom of which was made into two tongues which were fitted into the bolts. All these were abandoned for different reasons, chiefly, however, from the difficulty of side drainage, and it is possible these difficulties may be overcome by the proposed arrangement.

E. E. Russell Tratman.—Mr. Clarke refers briefly to the use of metal tie-plates between the rail and tie, and considers the joint to be the weak part of a track on cross ties. I think tie-plates are a very valuable means of improving the present tracks, until we are prepared to make a more decided improvement, and they are already being quite extensively used in this country and abroad. As to the joints, while there are various types and patents on the market and in use, I think a joint that will be really efficient under heavy traffic on a track laid with cross ties can be, but has not yet been designed. While I agree to some extent with Mr. Shinn as to the advantages of an absolutely rigid track,

based on the idea of smooth wheel rolling on a glass plate, I think the attainment of such a track very improbable, as it would involve much difficult work and heavy expenditure in general construction, apart from the track itself. The rail joint would be a difficult feature to manage, and even slight inequalities in track or rolling-stock, due to uneven wear, etc., would probably result in a rough riding road. A great increase over the present average degree of rigidity may, however, be practically attained by the construction of a more rigid track, particularly upon the already well consolidated roadbeds of the leading trunk lines. The rail joint question then again becomes important, in the endeavor to secure evenness of surface and consequent easy riding of the cars. The idea of the rigid railway is old. Mr. George Stephenson had one idea of fastening the rails down to solid rock, and he tried the plan in a rock cut on the Manchester and Leeds Railway, of which he was appointed engineer in 1839. According to published accounts the solid rock was dressed to a surface and the chairs were spiked directly to it, but the road was so rigid "that if a train passed over it at more than a walking pace, rails and springs were broken and in less than three weeks from the opening of the railway, orders were given for the rails to be taken up and placed upon sleepers in the ordinary way." The fastenings, however, probably worked loose, causing the battering of the rails between the wheels and the rock, and making a very rough road to travel over. If the rails could have been so securely and firmly fastened down as to become practically a part of the rock, which would have given absolute rigidity, it is possible that the results might have been somewhat different. Mr. Brunel originally designed the track for the Great Western Railway to consist of longitudinal timbers resting on transverse timbers placed upon piles, the piles being arranged in pairs at intervals of 15 feet. This was tried in 1838–39 between London and Maidenhead, but the ballast continually subsided, making a very elastic track with rigid supports at intervals of 15 feet. This track was soon taken up and its failure is generally attributed to its rigidity. Mr. Hartley built masonry walls upon which the rails were bolted, but "rails, tires and springs were broken daily and the plan proved an entire failure." On many of the old English railways stone blocks were used to support the chairs in which the rails rested, and it is stated that the riding over these blocks was harder than over wooden ties. This was probably due to the alternate elasticity and rigidity, and the perfectly natural result would be a more marked change from the suspended part of the rail to the rigid stone support, than to the semi-elastic wooden support. In some cases in India, where cast-iron bowl ties were laid in shallow ballast in rock cuts, it is said that the track was found to be too rigid, and it became necessary to put a greater depth of ballast under the ties. Mr. C. E. Stretton, an English engineer, makes the following remarks in his book on "Safe Railway Working:"

"Nothing can be worse than a rigid permanent way, but in the early days of railways this fact was not known or understood, consequently very many ideas and inventions proved failures. They provided a very strong road, but the rigidity was so great that the permanent way and rolling-stock were jarred to pieces, not worn out by ordinary working, thus clearly showing that a certain amount of elasticity was absolutely necessary. * * * Permanent way must be strong and firm, but at the same time possessing a certain amount of elasticity. It is very necessary that the elasticity should be uniform throughout, and not a system of alternate elasticity and rigidity, in which it serves to aggravate the rigidity by causing a succession of jumps and jacks." It may be, however, that if the rigidity was uniform throughout, the result would be even better.

In regard to experience with rigid road-bed and track on American railways, Mr. Charles Francis Adams has described a portion of the original track of the Boston and Lowell Railroad (opened on June 27th, 1835) as having the stone blocks, in which were inserted the oak plugs to which the rails were spiked, laid on a continuous foundation of parallel dry stone walls in trenches under each line of rails. These walls were $1\frac{1}{2}$ feet wide and from 2 to $4\frac{1}{2}$ feet deep. This construction did not prove successful, and it is stated that the directors "gradually learned to their surprise that speed without elasticity is always costly." A portion of the track of the old Columbus Railroad, of Pennsylvania, consisted of iron strap rails on continuous stone stringers 1 foot square, each line of stringers resting on a continuous bearing of broken stone filled into a trench.

James Owen, M. Am. Soc. C. E.—I noticed three or four weeks ago that the Great Western Railroad still adheres to the longitudinal stringers on their track; that was from London to Bristol.

C. E. Goad, M. Am. Soc. C. E.—That statement is true. Only three months ago I went over it. That portion of the Great Western passes mostly through an agricultural district. The gauge is still 7 feet, although there is a third rail for trains of ordinary gauge, and next year they will abolish the 7-foot gauge. The reason for keeping it is said to be that it is the only way they can compete with the South Western Railway to Exeter, which has a route 17 miles shorter. It always seems to me the longitudinal system of stringers has not had its fair trial, from the fact that a leading railway in England has for so long a time successfully maintained a long distance with this system.

E. P. North, M. Am. Soc. C. E.—Something was said about bolting a rail to solid rock. Possibly that was a mistake, as it would be almost impossible to find solid rock continuously. On the railroad from Albany to Schenectady, built by Mr. John B. Jervis about 1831, the rails were laid on longitudinal timbers which were fastened to blocks of limestone about 2 x $2\frac{1}{2}$ feet in place and rather over 1 foot thick. The longitudi-

nals were fastened to the limestone blocks by cast-iron angles; these were spiked into wooden trenails in holes drilled for them and to the sides of the timbers. I think the whole distance from Albany to Schenectady was laid with these blocks, and a part of the road from Schenectady to Saratoga. These were all taken out about 1845, as the timbers were battered to pieces under the locomotives, which did not weigh more than 5 tons. They were replaced by pieces of plank 3 inches thick and about 3 feet long. These blocks, I think, were used in Pennsylvania with the same result.

Mr. TRATMAN.—In regard to Mr. Stephenson's experiment, it is only a matter of memory, but I think it was tried as an experiment only in the rock cutting near Liverpool.

P. F. BRENDLINGER, M. Am. Soc. C. E.—Mr. Shinn spoke about having a good rigid roadway. I have wondered how we are going to get it. We would have to change our method of building railroads. If you take out a cut what will you do with the material? It is put in embankments. There are plenty of embankments 50 or 60 feet high, often more. We always allow in construction a certain percentage for shrinkage, and the banks keep on shrinking for years, as every superintendent of a railroad knows, and he has to keep raising the track all the time. How can there be a rigid substructure in a case of that kind?

HUNTER McDONALD, M. Am. Soc. C. E.—It strikes me that in some instances that difficulty may be overcome with piles. Piles shod with iron could be driven through a loose rock embankment to the original surface. There is one point that has probably escaped the attention of members, and that is the advantage that cross-ties offer in case of derailment. Trucks sometimes leave the rails and run on the ties for miles without accident, but should this occur where the track was laid on longitudinal stringers, such derailment would be likely to overturn the train.

J. FOSTER CROWELL, M. Am. Soc. C. E.—I regard this paper of Mr. Clarke as both valuable and timely. Owing to the causes which the author has so tersely and comprehensively enumerated, the era is at hand when railway companies will be brought face to face with the absolute necessity for providing tracks of adequate strength and endurance to withstand the exigencies not only of the present conditions of traffic, which have, in many cases, almost reached the proper limitation of the modern track structure, but of the inevitable future increase in loading and speed which economy in operating the railroads demands. The civil engineer cannot stop the process of evolution that applies to traffic movement, and must meet and anticipate its requirements for satisfactory structures. It is quite apparent, and could be demonstrated by a simple presentation of the statistics of the operation of railways with large traffic, that the excessive cost of maintenance of way represents a very large sum in the aggregate which, if capitalized, would justify a much greater

outlay for track than railroad managers venture to expend in first cost; and the conjectural advantages of additional safety, fewer wrecks and less wear and destruction of rolling stock can be estimated at perhaps almost an equal figure.

Moreover, if a track can be constructed of such perfection that the heaviest trains can be of maximum loading, moved over it at twice the present attainable speed for that class of trains, the sum required to build an additional track of the present type could be added to the improvement account without transgressing the laws of true economy. I believe that a longitudinal bearing system, in some form or other, is the most promising solution of the problem, but am not prepared to agree that independent bearers, connected flexibly, will afford the full measure of security and permanency required in a typically perfect track. Continuously longitudinal support for the rails is essential, but there should be in addition a rigid supporting lateral system on most road-beds in all exposed situations and especially on curves; and first of all the road-bed itself should be of an enduring and unchangeable type, not affected by water, frost or wear. The present road-bed, at its best, is an anomaly; what would be thought of the wisdom of erecting a permanent building upon a foundation requiring the unceasing efforts of a standing army of workmen to make it serve its purpose? And yet this is what has to be done, yesterday, to-day and forever on every railroad in this country; and despite this vast expenditure of costly effort, each advance in volume and weight of traffic only brings the structure nearer to the point of failure. I do not think that any system of track structure that can be devised can be rendered effectual and economical on present road-beds, and that any new system should include the substructure.

T. C. Clarke, M. Am. Soc. C. E.—In replying to the discussion, I would say I am much gratified at the interest shown by members of the society in the important subject of my paper, and their belief that there is room for improvement of our tracks. Mr. H. G. Prout is mistaken in supposing that I recommend the Hohenegger system, or any specific system. My intention in this paper was to indicate certain general lines of investigation. The point referred to by Mr. Prout, of insufficiency of bearing surface, is one of such great importance, and its evil effects, as shown by Mr. Shinn, from the experiments of Dr. Dudley, are so conspicuous that I will here go much into detail. My observations show that an 80 pound rail will distribute the weight of one 8-ton wheel over two ties placed 2 feet apart; thus each tie carries 4 tons. This load does not sink the ties into the ballast over one-eighth of an inch, which corresponds to a grade of 13 feet per mile. These ties have 8 inches face, by a length available for bearings on each side of 2 feet, or 4 feet in all, giving a bearing surface of 2.64 square feet. The ties being 2 feet apart between centers, we have a bearing area per lineal foot of rail of 1.32 square feet. As some wheels carry more than 8 tons I have

assumed 1½ square feet as the least allowable bearing surface per lineal foot of rail of a stiffness equal to 80 pounds section. This is equal to a pressure of about 1½ tons per square foot of bearing surface with the wheel loads now in use.

I know of but three modes of supporting railway rails: by cross-ties, by the pot system and by longitudinal bearers. It may be interesting to compare the dimensions and weights of tracks on these three systems, worked out on the basis of allowing a bearing surface of 1½ square feet per lineal foot of rail.

These systems are all metallic, and to give as good a grip upon the ballast as the standard wooden cross-tie system has, the cross-ties, pots, and longitudinals must have flanges running down at least 6 inches into the ballast. The cross-tie system has ties 9 feet long by 12 inches bearing surface, which are placed 2 feet 8 inches between centers. These ties would average 300 pounds each, if of metal one-half inch thick; and the weight of metal per yard would be:

	Pounds.
Per yard	333
Two 80-pound rails	160
Per linear yard	493

The pot system would have stamped plates 2 feet 2 inches square by one-half inch thick placed 3 feet apart between centers, united by light cross-ties, weight—

	Pounds.
Two pots, 120 pounds	240
Tie	10
Two 80-pound rails	160
Per linear yard	410

The longitudinal bearers would be 18 inches wide by one-half inch thick, united by light cross-ties:

	Pounds.
Two bearers, 120 pounds	240
Tie	15
Two rails	160
Per linear yard	415

This track would have weight enough, bearing enough, and grip enough to lie still and not deflect under passing loads more than enough to give a 13-foot grade per mile.

Many inventors are striving to produce a cheap metallic cross-tie system with ties weighing under 150 pounds each of 6-inches face, 7 feet long and placed 3 feet apart. Such a track would weigh—

	Pounds.
Ties	150
Rails, two of 80 pounds	160
	310

per linear yard, or much less than any as above described. How is this economy obtained? By cutting down the bearing surface to one-half a square foot per linear foot of rail, or one-third of what it ought to be, thus increasing the subsidence of the ties into the ballast and increasing the grade of the hollow, out of which the locomotive is always striving to climb, to 40 feet per mile. To avoid this, rails would be required of a section weighing 120 pounds per yard, and thus would take away the supposed economy.

Referring to the views of Mr. Shinn, who thinks that the foundations of rails should be rigid and unyielding, it seems only necessary to point out that wheels cannot be maintained truly circular. Therefore shocks and blows must take place between wheel and rail. If the latter is unyielding it would be battered down and the wheels also would suffer. But by a little yielding, not over one-eighth of an inch per tie, or about 13 feet per mile, they would cushion these blows entirely.

Finally, tracks made heavy enough to give the necessary bearing of 1½ square feet per lineal foot of rail would cost double as much as our present wooden cross-tie system does. But the locomotive would haul more cars and at a higher speed, and owing to its greater strength and the absence of wooden material, the saving of cost of repairs would more than recoup the interests on the increased cost, which at five shillings would be $385 per mile per year.

ERRATA.

Page 136, second line from bottom, Plate VII should read Plate XVIII.

Page 235, fourth line from bottom, Plate LII should read Plate XLII.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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(Vol. XXV.—September, 1891.)

YIELD OF THE SUDBURY RIVER WATER-SHED IN THE FRESHET OF FEB. 10TH–13TH, 1886.

By DESMOND FITZGERALD, M. Am. Soc. C. E.

One of the most disastrous rainstorms that has visited New England since accurate records have been kept, occurred in February, 1886. About 8 inches of rain fell over the greater part of the State of Rhode Island, which included the center of the storm. In Massachusetts the rain was less severe, although even in this State a great deal of damage was done. It is obvious that had the heaviest part of the precipitation fallen on the upper portions of the rivers instead of near the sea, the effects would have been far different.

Professor Winslow Upton, of Providence, estimated the areas covered by different amounts of rainfall as follows:

AMOUNT OF RAINFALL.	AREA IN SQUARE MILES.
Over 8 inches.	750
Between 7 and 8 inches,	750
" 6 " 7 "	1 500
" 5 " 6 "	1 850
" 4 " 5 "	2 750
Total above 4 inches,	7 600

A considerable amount of snow and ice covered the ground at the beginning of the rain, averaging, according to general testimony, about 2 inches of melted snow. Some of the rain as it first fell was absorbed by the snow. The temperature during this portion of the storm was near the freezing point, so that the snow melted slowly, but perhaps three-quarters of it went off with the rain, and the remaining portion followed a few days after the end of the rain. This kept up the flow of the streams in a remarkable manner long after the rain had ceased, and was one of the characteristics of the freshet. The ground was frozen, and when it is in this condition, the frost acts like a waterproof cover over the surface.

The rain in the Sudbury River water-shed began at 7 P.M. of the 10th, and ended at midnight of the 13th, thus extending over three days. The amount of rainfall was 4.64 inches, which with the melted snow made 6.64 inches total precipitation. The greater part of the rain fell during the afternoon and night of the 12th. The rise of temperature, which aided in carrying off the snow, is shown by the daily maxima of temperature observed. On the 11th and 12th the thermometer registered 37 degrees Fahr., but rose to 46 degrees on the 13th, 51 degrees on the 14th, and 54 degrees on the 15th and 16th. On the morning of the 16th the snow had all disappeared with the exception of a little in the woods.

The writer has been unable to find a continuous record of the rain on the Sudbury water-shed, but from notes and comparison with records in other places, he is led to believe that the curve of rainfall as assumed and drawn on the accompanying diagram is quite near to the true distribution of the rain. At Hopkinton 4.76 inches, and at Lake Cochituate 4.95 inches, were reported. The former is within the area which we are discussing and the latter contiguous to it. These observations confirm the accuracy of the total rainfall, as determined by the rain gauge at Framingham.

It is unnecessary to discuss the value of accurate river gaugings in time of freshet flow. The hydraulic engineer has frequent occasion to use such data in the course of his practice. It is to be regretted that more information of this kind has not been published.

The Sudbury River water-shed is one of the sources of water supply of the City of Boston. It is 25 miles west of the city. Its area is about 75 square miles, varying slightly from this amount, as is shown

Yields of Sudbury River and Cold Spring Brook Water-sheds during the Freshet of February, 1886.

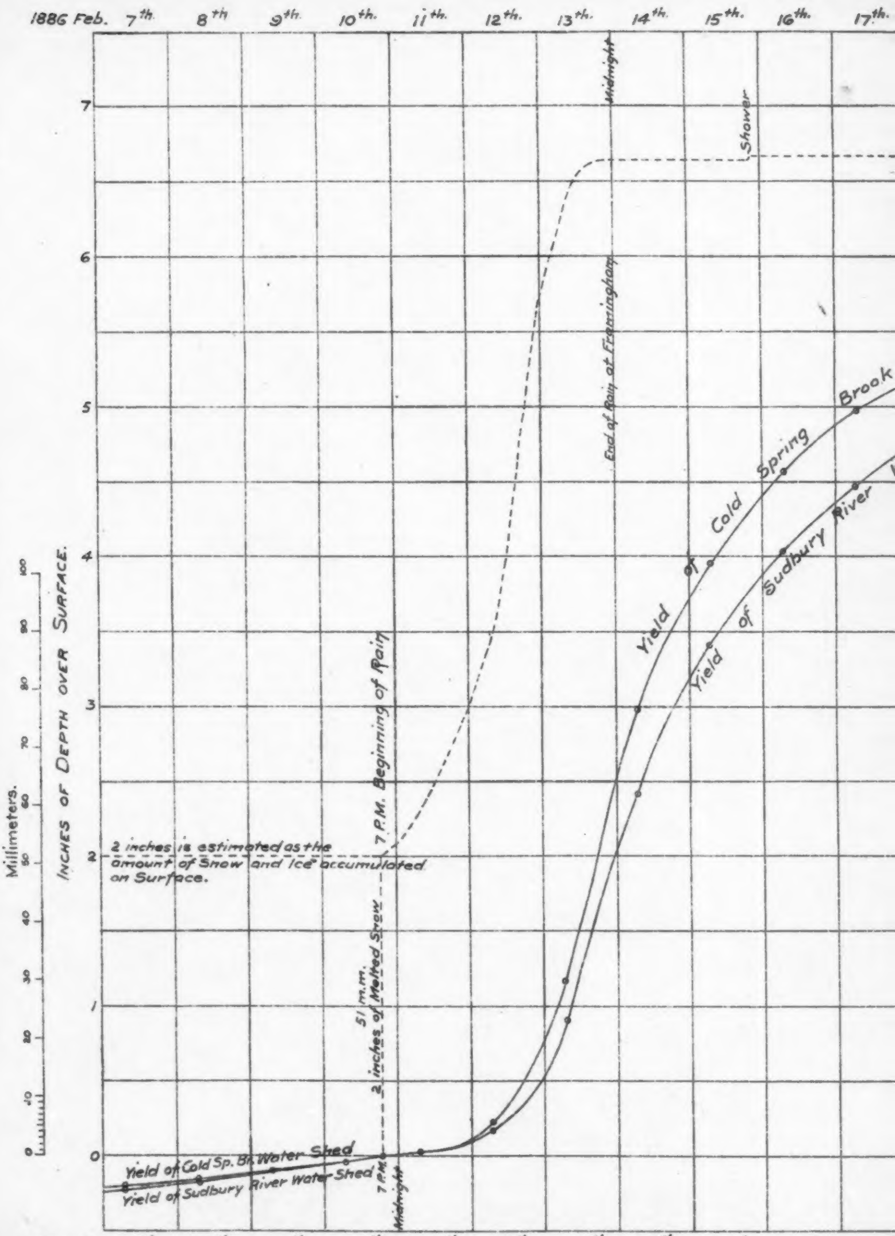
	Sudbury River Water-shed. Area = 74.656 square miles......13th to 18th. " 75.199 " " .... { 6th to 12th inclusive. 19th to 24th "				Cold Spring Brook Water-shed. Area = 6.434 square miles.				
	Yield in gallons.	Yield in gallons per sq. mile.	Inches in depth on water-shed.	Points for cumulative Profile.	Yield in gallons.	Yield in gallons per sq. mile.	Inches in depth on water-shed.	Points for cumulative Profile.	Remarks.
February 6..	79 140 000	1 052 000	0.061	—0.287	6 300 000	979 000	0.056	—0.2615	On February 13th the yield of the Sudbury River water-shed was as follows: Seven A.M. to noon, at rate of 2 136 million gallons in 24 hours. Noon to 4 P.M., at 2 060 millions in 24 hours. 4 P.M. to 7 P.M., at 2 087 millions in 24 hours. 10 P.M. to midnight, at rate of 1 997 millions in 24 hours. Midnight to 6 A.M. on the 14th, at rate of 1 647 millions in 24 hours. Rainfall—7 P.M., Feb. 10th, to midnight, Feb. 13th, 1886.. 4.64 inches. Snow on ground...... 2.00 " Total rain and snow.. 6.64 " Shower of Feb. 15th, 8.30 P.M., 0.02 Feb. 19th, 4 P.M. to midnight, 0.66
" 7..	78 240 000	1 040 000	0.060	—0.226	5 880 000	914 000	0.053	—0.205	
" 8..	81 090 000	1 078 000	0.062	—0.166	5 880 000	914 000	0.053	—0.152	
" 9..	92 220 000	1 226 000	0.071	— .104	7 270 000	1 130 000	0.065	—0.099	
" 10..	86 180 000	1 146 000	0.066	{ — .033 * 0.000 0.033 }	7 720 000	1 200 000	0.069	—0.0345	
" 11..	170 740 000	2 270 000	0.131	0.164	20 980 000	3 261 000	0.188	{ 0.0000 * 0.0345 0.222 }	
" 12..	943 210 000	12 543 000	0.722	0.886	106 140 000	16 496 000	0.949	1.171	
" 13..	1 997 550 000	26 757 000	1.540	2.426	201 380 000	31 299 000	1.801	2.972	
" 14..	1 289 600 000	17 274 000	0.994	3.420	109 240 000	16 979 000	0.977	3.949	
" 15..	838 210 000	11 228 000	0.646	4.066	69 090 000	10 738 000	0.618	4.567	
" 16..	531 120 000	7 114 000	0.409	4.475	46 050 000	7 157 000	0.412	4.979	
" 17..	428 510 000	5 740 000	0.330	4.805	27 020 000	4 200 000	0.242	5.221	
" 18..	357 190 000	4 784 000	0.275	5.080†	23 390 000	3 635 000	0.209	5.430‡	
" 19..	395 550 000	5 260 000	0.303	5.383	32 920 000	5 117 000	0.294	5.724	
" 20..	336 430 000	4 474 000	0.257	5.640	24 450 000	3 800 000	0.219	5.943	
" 21..	269 270 000	3 581 000	0.206	5.846	24 410 000	3 794 000	0.218	6.161	
" 22..	265 020 000	3 524 000	0.203	6.049	15 750 000	2 448 000	0.141	6.302	
" 23..	168 120 000	2 236 000	0.129	6.178	15 050 000	2 339 000	0.135	6.437	
" 24..	172 720 000	2 297 000	0.132	6.310	11 820 000	1 837 000	0.106	6.5435	

The Sudbury River water-shed includes that of Cold Spring Brook.

*Beginning of rain = 0.000. †5.08 inches collected. ‡5.43 inches collected. 6.66 total rain and snow.

FRESHET OF FEBRUARY

Cumulative Diagram of Simultaneous Precipitation and Yield of Sud



ESHET OF FEBRUARY 10TH.-13TH. 1886.

n and Yield of Sudbury River and Cold Spring Brook Water

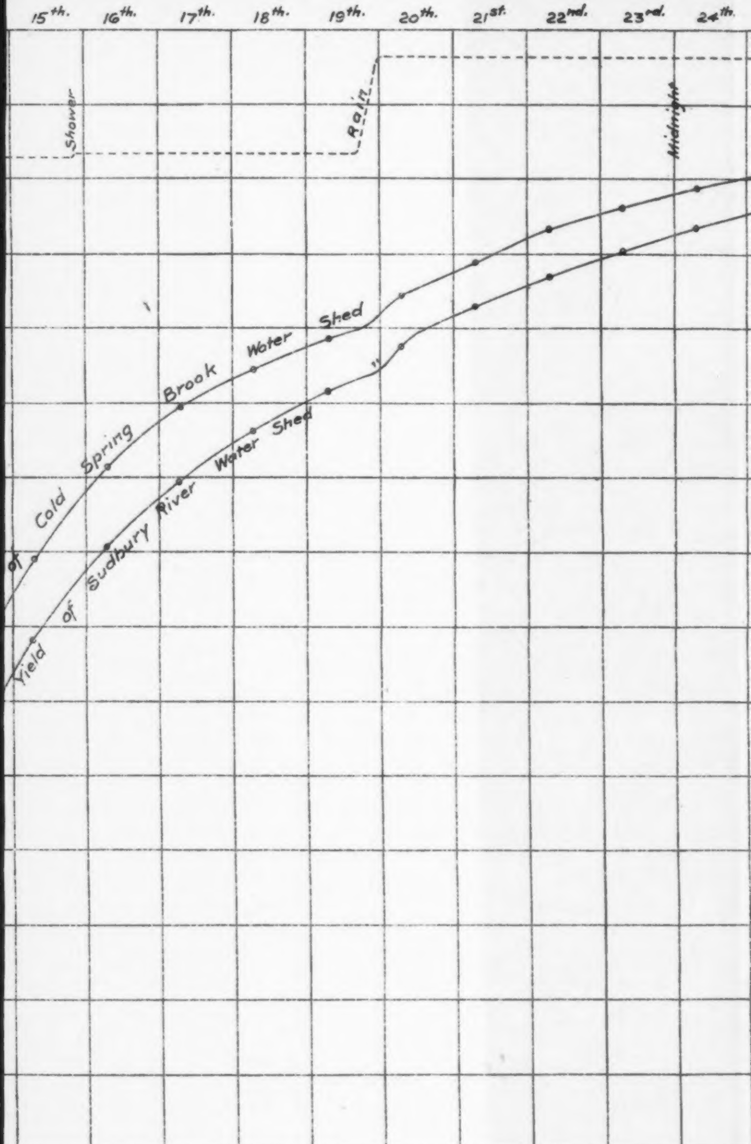


PLATE LVI
TRANS. AM. SOC. CIV. ENG'RS.
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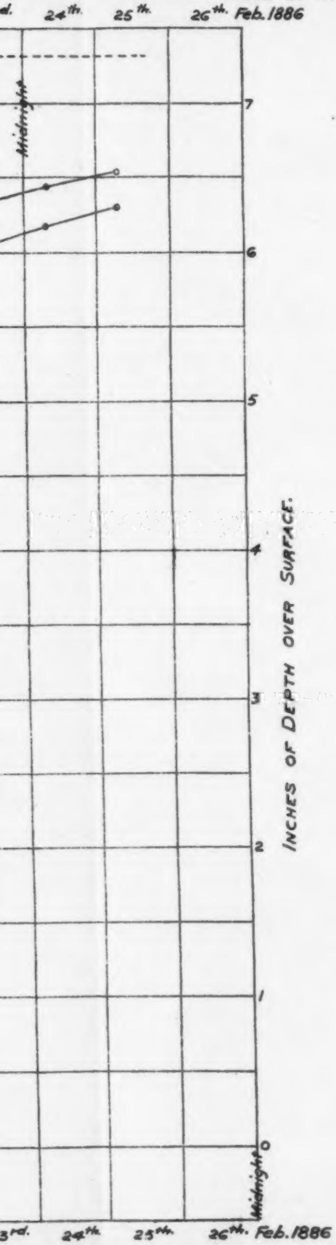
FITZGERALD ON
YIELD OF THE SUDBURY RIVER.

k Water Sheds.



in the accompanying table. The topography represents a fair average New England water-shed. About 38 per cent. of the surface is covered with woods. The Cold Spring brook water-shed forms a fraction of the above area. It covers 6.434 square miles. The boundaries of both water-sheds have been determined by survey. The gaugings are made daily at 7 A.M. The yields are determined by the flows over the dams at the outlets of the respective water-sheds, modified by the gains or losses in the storage basins upon their surfaces. The accompanying table gives the yields in gallons; the yields per square mile in gallons; these yields reduced to their equivalent depth in inches upon the water sheds; and in the final column in each case, the successive additions from the time when the rain began, the latter point being taken as zero. The yields preceding this time are tabulated as minus quantities in order to separate them from the total collections which follow and which are due to the rain.

The writer has plotted three lines upon the diagram which accompanies this paper. The upper line is the rainfall. It has been started 2 inches higher than the lines which are about to be described, to allow for the 2 inches of snow upon the ground at the beginning of the rain. The other two lines represent the yields of the two water-sheds in inches in depth, and starting from zero at the beginning of the rain they represent at any point the total yield at that point reckoned from the beginning of the rain. It will be noticed that the yields previous and subsequent to the rain were small, and they are shown in the diagram by the flat inclination of the lines of flow as they approach the storm; consequently the large flows were due entirely to the particular storm we have been discussing. For this reason it is safe to assume that out of the 6.66 inches total rain and melted snow came the 5.08 inches on the Sudbury, and 5.43 inches on the Cold Spring brook water-sheds which were collected between the beginning of the rain and the morning of the 19th, five days after the rain ceased.

It will be noticed that the maximum yield per 24 hours on the Sudbury water-shed was equal to 1.54 inches in depth upon its surface, and on Cold Spring brook water-shed to 1.801 inches. The maximum rate of yield on the larger water-shed occurred between 7 A.M. and noon of the 13th. For this period the flow averaged at the rate of 2 136 000 000 gallons in twenty-four hours, or 1.646 inches in depth on the water-shed per day. On March 26th, 1876, on the Sudbury River water-shed, almost

exactly the same yield was gauged, viz., 1.559 inches in depth per twenty-four hours on the water-shed.

The dams on the Boston Water Works are proportioned for a continuous flow of 6 inches depth per day over their water-sheds. This is not an unreasonable provision for maximum flow, as will be apparent on careful consideration. Mr. James B. Francis, M. Am. Soc. C. E., has given this Society the results of his investigation of the great storm of October, 1869,* when 12.35 inches fell at the center of the storm, in Connecticut. At Middleton, 7.15 inches fell in twenty-four hours, and it is possible that at the center of the storm considerably more than 8 inches fell in one day. If such a rain as this should fall on frozen ground covered with a body of snow, and accompanied by a high temperature, it might create a flow equal to 5 or 6 inches in depth upon a water-shed in twenty-four hours.

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* *Transactions*, Vol. 7, page 224. August, 1878.